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STRUCTURAL EVALUATION OF OPEN-GRADED BASES FOR HIGHWAY PAVEMENT--ETC(U)
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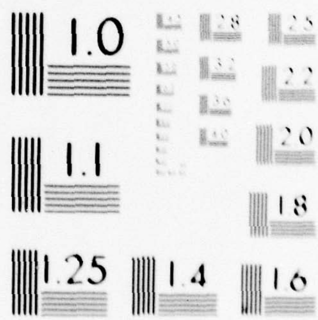
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STRUCTURAL EVALUATION OF OPEN-GRADED BASES FOR HIGHWAY PAVEMENTS

by

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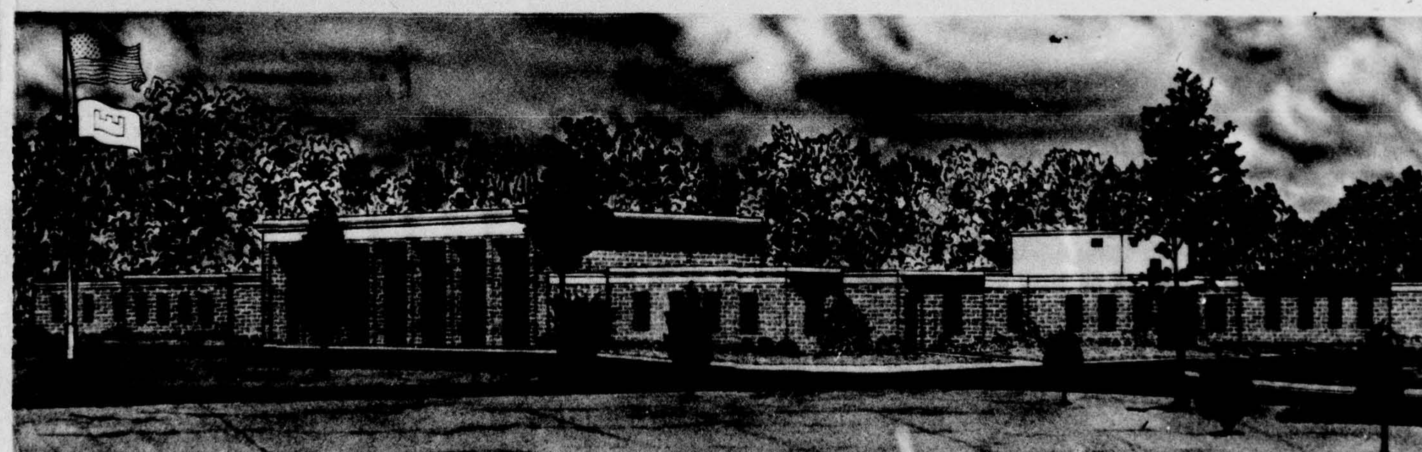

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Base courses
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Triaxial shear tests

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Gyratory shear tests and repeated load triaxial compression tests were conducted on five different base courses furnished by the New Jersey Department of Transportation (NJDOT). Two of the materials, one stabilized with asphalt and the other unstabilized, were open-graded aggregate bases designed to provide a high degree of porosity. The other three materials were conventional bases currently being used by the NJDOT. The conventional bases were a high-quality, asphalt-stabilized base material, a crushed rock, and a pit-run gravel.
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20. ABSTRACT (Continued).

The laboratory tests indicated that for dynamic loads, such as would be the case for high-speed highway traffic, the open-graded bases would perform better than or as well as the nonstabilized conventional bases but not as well as the high-quality stabilized base. For static loading, the nonstabilized bases were superior to the asphalt-stabilized bases. The minimum coverage requirement for the open-graded bases is estimated to be approximately 6 in.

The data provided by gyratory testing proved to be useful in evaluating the base materials, and the recommendation is made to continue development of gyratory testing for evaluation of pavement materials.

PREFACE

The investigation reported herein was conducted by the U. S. Army Engineer Waterways Experiment Station (WES) under joint sponsorship of the Office, Chief of Engineers, U. S. Army (OCE), and the New Jersey Department of Transportation (NJDOT). OCE sponsorship was authorized under the Military Construction RDT&E Program, Project 4A762719AT40, "Pavements, Soils, and Foundations," Task A2, Work Unit 004. NJDOT sponsorship was authorized by FY 77 Agreement No. WES-77-02, State of New Jersey, dated 3 May 1977, signed 8 June 1977 (New Jersey State Project No. 7740). The New Jersey Department of Transportation received partial funding from the U. S. Department of Transportation, Federal Highway Administration. The study was conducted during the period June 1977 to September 1978.

The gyratory shear testing was conducted and the test data were reduced by personnel of the Pavement Materials Research Facility, Geotechnical Laboratory (GL), under the supervision of Mr. T. D. White. A data report on the gyratory shear test was prepared by Mr. L. N. Godwin. The repeated load triaxial tests were conducted and the data reduced by the Soils Research Facility, GL, under the supervision of Mr. V. H. Torrey III. The data report for the repeated load triaxial tests was prepared by Mr. R. D. Barnette. The conduct of the investigation was under the general supervision of Mr. H. H. Ulery, Jr., Chief, Pavement Design Division, and Mr. J. P. Sale, Chief, GL. The data were analyzed and the report was written by Dr. W. R. Barker and Mr. R. C. Gunkel, both of the Pavement Design Division, GL.

NJDOT personnel involved with the project were Mr. K. C. Afferton, Mr. G. S. Kozlov, and Mr. B. Cosaboom.

COL J. L. Cannon, CE, was Commander and Director of the WES during the conduct of the study and the preparation of this report. Mr. F. R. Brown was Technical Director.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
gallons (U. S. liquid)	3.785412	cubic decimetres
inches	25.4	millimetres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.45359237	kilograms
pounds (force)	4.448222	newtons
pounds (mass) per cubic foot	16.01849	kilograms per cubic metre
pounds (force) per square inch	6894.757	pascals
square feet	0.09290304	square metres
square yards	0.8361274	square metres
tons (2000 lb, mass)	0.90718474	kilograms

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

STRUCTURAL EVALUATION OF OPEN-GRADED BASES FOR HIGHWAY PAVEMENTS

PART I: INTRODUCTION

1. Evaluating the performance potential of pavement materials without the benefit of performance data from realistic test sections or actual in-service pavements is a major deterrent in the development of innovative pavement systems. The principal factor motivating researchers toward the development of theoretically based design procedures is the desire to take advantage of new concepts in pavement construction. Despite this factor, the methodology for theoretically predicting performance of pavement systems containing granular structural elements is almost totally nonexistent. Thus, when the question arose of how to improve the internal drainage of pavement systems, a methodology could not be found, within price constraints, for evaluating the structural performance of highly porous materials required to provide drainable bases.

Background

2. The U. S. Army Corps of Engineers has long recognized the problem of subsurface drainage and has published Technical Manual 5-820-2, "Subsurface Drainage Facilities for Airfields." The criteria for base course drainage were developed by Professor A. Casagrande based on a subsurface drainage study conducted by the U. S. Army Engineer Division, New England, and its Boston District during the period 1945-1947. Methods developed by K. Terzaghi for design of filter courses around subdrains were later modified as a result of investigations conducted at the U. S. Army Engineer Waterways Experiment Station (WES). An additional study, reported in WES Technical Report No. 3-786, July 1967, entitled "Drainage Characteristics of Base Course Materials, Laboratory Investigation," states that recent observations indicate the base

courses and filter courses are periodically saturated at certain airfields and some extensive subdrainage systems appear to be largely ineffective. In the study, it was concluded that it is theoretically possible for a base course material to meet drainage criteria in Technical Manual 5-820-2 and still remain nearly 100 percent saturated. Even when a material contains as little as 5 percent fines, the effective porosity can approach zero. It is apparent that to obtain highly porous bases, materials not meeting present gradation requirements for bases must be employed.

3. At the University of Illinois at Urbana, Illinois, Barenberg and Tayabji[1] conducted a laboratory experiment aimed at evaluating the performance of an open-graded, hot-mixed bituminous aggregate mixture (OGBAM) as a porous base for pavement drainage. In the experiment, a circular test track was used to test six different pavement systems employing OGBAM bases. Dynamic loading was applied to the test pavements, and water was passed through the OGBAM drainage layers to simulate surface and lateral infiltration. The results of the experiment indicate that the OGBAM possesses a very high order of permeability, and considering the severity of the loading, performed adequately as a structural base. It was interesting to note that at the end of the experiment the OGBAM in the wheel path was loose and in the state of a cohesionless granular material.

4. The New Jersey Department of Transportation (NJDOT) recently initiated a study of rapidly draining bases that is to culminate in the field testing of pavement systems employing such bases. In this study, the NJDOT has set drainage criteria wherein the base material must be drained within hours as opposed to 10 days for drainage allowed by the Corps of Engineers criteria. The rapid drainage criterion was established to prevent freezing of water in base courses by falling temperatures that usually follow wintertime rains. Such rapid drainage would require very highly effective porosities and necessitate the use of open-graded materials for which performance data are not available.

Need for and Purpose of Study

5. Prior to investing in expensive field testing or even the less expensive small-scale model testing, the NJDOT wished to conduct laboratory tests to evaluate the structural potential of a nonstabilized open-graded aggregate and a bituminous-stabilized open-graded aggregate to serve as bases for high-volume pavements.

6. Since WES is involved in research aimed at the development of more rational procedures for design of pavements and has had an ongoing project for characterizing pavement materials, WES felt it would be beneficial to both NJDOT and WES to jointly sponsor the laboratory evaluation of the NJDOT open-graded base materials.

7. Although WES has no funded study in base drainage, it has been previously recognized that the present gradation criteria for airfield pavements do not ensure a base of adequate drainage. Thus, in addition to material characterization, WES has an interest in the application of the results of the NJDOT study to the design of airfield pavements.

Scope of Work

8. The NJDOT selected five base course materials for evaluation in the test program. Three of the materials were standard base materials used by the NJDOT; the other two materials were open-graded bases. Laboratory tests were conducted to achieve a dual purpose: produce a relative evaluation between the different materials and provide strength parameters for each material. The relative comparison of the different materials provided a subjective analysis of pavements containing open-graded bases, whereas the strength parameters provided an analytical analysis. In part, funding and time restraints dictated the particular laboratory test and placed limitations on the number of tests to be conducted. A literature review was conducted to ensure that the laboratory tests selected would yield the desired data.

PART II: TESTS AND PROCEDURES

Selection of Test Methods

Literature review

9. In the development of a structural design procedure for flexible airport pavements, Barker and Brabston[2] chose not to specify laboratory testing for granular materials but to rely instead on gradation requirements for strength and charts for determining stiffness. Obviously, this limits the usefulness of the procedure, particularly in evaluating the performance of different granular materials. At the time of the development of the procedure, analytical models and laboratory test procedures were reviewed, and no combination was found that could be incorporated into a practical design procedure. It was felt that the behavior of granular materials is dictated by the fact that they are composed of separate discrete particles; therefore, modeling pavements containing these materials as layered-linear elastic continuums involves simplifications of such magnitude as to require an empirical approach to predicting performance.

10. Chou[3] conducted an extensive state-of-the-art review of the engineering behavior of pavement materials and of laboratory tests being conducted to characterize these materials. From Chou's report it is evident that the main thrust being made by researchers to characterize granular materials, both nonstabilized and bituminous-stabilized, is through the use of the repeated load triaxial test.

11. Even though the literature is full of research being conducted to quantify the engineering behavior of pavement materials, a realistic methodology still does not exist for predicting the performance of granular materials in pavement systems. Barksdale[4] used the repeated load triaxial test to measure permanent deformation and a system for rating of the different materials relative to each other. To evaluate the effect of fines on the behavior of granular base materials, Ferguson[5] also used the results of repeated load triaxial tests to make relative comparisons of bases having different fine contents. From the

study made of the literature, it was concluded that for the requirement of the NJDOT, primary reliance should be placed on a relative evaluation of different base materials.

Repeated load triaxial test

12. It was believed that such an evaluation could best be accomplished by conducting repeated load triaxial tests of the NJDOT open-graded base and for comparison standard base materials. When considering the number of materials, the different states of stress, and the different temperatures for the bituminous-stabilized materials, the total number of triaxial tests required for the study would be prohibitively costly. Also in the WES material characterization study, one objective has been to identify a laboratory test that could be conducted as a routine material test without the requirement of highly skilled laboratory technicians. Thus, although it was felt that the repeated triaxial was the laboratory test most suited for evaluating the materials, there were motivating factors for considering an alternate test.

Gyratory shear test

13. In seeking an alternate test for the repeated load triaxial test, it was suggested that the use of the gyratory shear test be considered. Although the concept of gyratory testing originated in the Texas Highway Department,[6] the major development of the gyratory testing machine (GTM) has been at WES.[7,8] Early research with the GTM has been in connection with compaction of soils and bituminous materials. Mr. J. L. McRae developed an equation for computing the shear stress within the material during gyratory testing. Based on the computed shear stresses, McRae[9] advocates the use of the GTM as a means of evaluating the strength parameters of pavement materials. In addition to strength data, McRae computes a gyratory shear modulus that is a measure of stiffness. At the present, the GTM is marketed by SOIL TEST, Inc., who now publish the instructional manual prepared by McRae,[9] which provides instructions for conducting the gyratory shear test. Research projects have been conducted by Parker[10] and by Wahls[11] that involved measuring strength parameters of soils using the GTM. In both studies, it was concluded that the gyratory shear test provides data indicative of the shear strength.

14. If the information provided by gyratory testing could produce the necessary material evaluation, then this particular test has several advantages over other laboratory tests. The test is easy to conduct and would be relatively inexpensive. The state of stress and material temperature can be varied with relative ease. In addition to strength and stiffness, information concerning density requirements can be obtained with almost no extra effort. Considering the very favorable results obtained in previous studies using the GTM, the decision was made to conduct the evaluation with the gyratory shear test as the primary procedure but with a limited number of repeated load triaxial tests being conducted for verification.

Experimental Test Program

Test materials

15. To accomplish the primary objective of the study, gyratory shear and repeated load triaxial tests were conducted on both nonstabilized open-graded (NSOG) and bituminous-stabilized open-graded (BSOG) base materials, and on a dense-graded bituminous-stabilized base course (BSBC) material and a dense-graded crushed-stone base course material designated NJDOT base "5A." In addition, gyratory shear tests were also conducted on a bank-run material designated as NJDOT base "1A." All materials and the gradations of the materials tested, including the bitumen for stabilization, were obtained from the NJDOT. The description of the materials and details of the sample preparation are contained in Appendix A.

Test procedures

16. Gyratory shear tests. The basic procedure used in the gyratory shear testing is given in American Society for Testing and Materials (ASTM) Method D 3382.[12] In the gyratory shear testing, each sample was tested at gyratory angles of 0.3, 0.7, and 1.1 deg* and at applied

* A table for converting U. S. customary units of measurement to metric (SI) units is given on page 4.

vertical pressures P_v of 25, 50, 75, and 100 psi for each gyratory angle. In addition, the complete series of tests for the bituminous-stabilized sample were conducted for temperatures of 75°F, 90°F, and 110°F. The tests necessary for making the corrections for machine error and wall friction were conducted as specified.

17. A description of the test and tabulated test data are contained in Appendix A.

18. Repeated load triaxial tests. The repeated load triaxial tests were conducted to determine the permanent deformation characteristics and resilient modulus for each of the materials tested. Available funds limited the triaxial testing to a single sample for each of four materials: I (BSBC), III (BSOG), V (base 5A), and II (open-graded unbound base). The test procedure employed was designed to obtain from a single sample a measure of permanent deformation characteristics for each material and the resilient properties as a function of state of stress. The details of the tests and test data are contained in Appendix B.

PART III: ANALYSIS OF TEST RESULTS

Material Characteristics

Material density

19. Considerable research[13-15] has been accomplished at WES concerning the compaction of pavement materials utilizing the GTM. One study[13,15] in particular dealt with materials very similar to the NJDOT bases 1A and 5A. The study indicated that the compaction effort used in the GTM for base 1A, i.e., 30 revolutions at a vertical pressure of 50 psi and an angle of tilt of 1 deg, should produce densities comparable to those produced by 56-blow impact compaction in materials similar to base 1A. For the base 5A, the indication is that the compaction effort used would produce densities somewhat less than those that would be obtained from 56-blow impact compaction. Thus, it was felt that the densities obtained in the GTM would be somewhat representative of densities that would be obtained by impact compaction. The results of gyratory compaction are given in Table 1. As can be seen from the results presented, the change in density of the base 1A with increased compaction effort was less than the change in density for either the NSOG base or the base 5A. If the GTM better represents field compaction, as claimed by some researchers, then densities specified by impact compaction will be much more difficult to obtain in the field for the base 5A than for the base 1A.

20. In compaction of the bituminous-stabilized samples, it was apparent that a higher compaction effort would be required to obtain a satisfactory sample for these materials than was used for compaction of the nonstabilized materials. The compaction effort, which produced samples in the expected density range and was used in preparing the sample, was 30 revolutions with a ram pressure of 200 psi and angle of tilt of 1 deg. This compaction effort is in agreement with the effort used in the study reported by Reference 8.

21. The densities obtained for the triaxial testing were comparable but slightly less than the densities obtained in the GTM. The compaction

of the nonstabilized samples for the triaxial testing was by impact, using a procedure to obtain densities close to the densities obtained in the GTM. The energy used in the compaction was not measured. For the bituminous-stabilized sample, a static compaction procedure was employed. Thus, little information is provided by the comparison of densities between the samples prepared in the GTM and the samples used in the triaxial testing.

22. The densities of the bituminous-stabilized materials and the NSOG material obtained in the WES testing were very close to the expected densities as furnished by the NJDOT. For the bases 1A and 5A, the densities obtained by WES, approximately 8 pcf for both materials, were much greater than the expected densities as furnished by the NJDOT. To check the densities, WES compacted a sample of base 1A according to the procedures of ASTM D 698-70.[16] The sample was to be compacted at a water content of 8.8 percent, which the NJDOT had indicated was the optimum water content. During the compaction, free water was squeezed from the sample; therefore, the final water content of the sample was less than the 8.8 percent. The density obtained in the test was 136.8 pcf, which agreed with the densities obtained by the GTM. No explanation can be offered for the difference between the densities obtained in the WES testing and the densities furnished by the NJDOT. The indication is that the WES densities do correlate closely with the ASTM D 698-70[16] densities.

Gyratory shear strength

23. The formulas developed by McRae[9] for computing the gyratory shear strength S_G* in psi when the GTM model BG-4C is used are as follows:

$$S_G = \frac{90_p - 2.55F + N \cdot b}{12.56h} \frac{\theta_{max}}{\theta_o}$$

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix C).

in the case of a 4-in. sample and

$$S_G = \frac{90_p - 3.82F + N \cdot b}{28.26h} \frac{\theta_{max}}{\theta_o}$$

in the case of a 6-in. sample where

p = gage pressure for upper roller

F = force caused by wall friction

N = normal vertical load on specimen

b = arm of vertical force couple = $h \cdot \tan \theta_o$

h = height of sample

θ_{max} = maximum gyratory angle

θ_o = initial gyratory angle

24. These equations were derived from simplified free-body representations of the GTM and were used in this study for computing the gyratory shear. As noted in the equation, the computations include correction for sidewall friction forces. This friction correction requires conducting special tests for measuring the sidewall friction for each sample at each vertical pressure. In addition, there is a machine correction that is determined by conducting gyratory shear for dry Ottawa sand. Parker[10] also included a correction for the weight of the GTM housing that surrounds the soil sample. This correction is probably indirectly taken care of by the machine correction and was not used in the WES study.

25. The results of the gyratory shear testing are given in Appendix A. The results, in regard to the computed cohesion, are an immediate reason for doubting the validity of the test data. The fact that negative values of cohesion (Table 2) are computed for the bituminous-stabilized materials and relatively large values are computed for the nonstabilized open-graded material is certainly sufficient evidence for discounting the values for cohesion.

26. It is apparent from the data that the side friction corrections cannot be applied to the gyratory shear in the manner indicated in the formula. Undoubtedly, in the gyratory test the full friction

resistance is not developed. To make a proper correction for the friction, a method would have to be devised for considering the relative movement of the sample with respect to the mold wall. Such consideration would appear to be so complicated as to be impracticable. Also, there appeared that little was gained in applying the machine correction to the computation. Plots in Appendix A (Figures A4-A13) provide the relationship between gyratory shear, as computed without the friction and machine corrections, and the applied vertical pressure. A summary of the test results is provided by Table 2 with more complete results in Tables A5-A9.

27. In comparing the behavior of the different materials, it is seen that the bituminous materials do not fare as well as would be anticipated. The NSOG base performed surprisingly well, and from the results of these tests, appears to be the best of the different materials. Ranking of the materials based on the gyratory shear strengths is as follows: materials II (NSOG), V (base 5A), IV (base 1A), III (BSOG), and I (BSBC).

28. Such ranking must be judged with regard to the procedures used in conducting the test. Of particular importance is the fact that the upper roller pressures were taken with a static loading, i.e., the GTM was stopped and the load was allowed to stabilize. Such a test procedure simulates a static loading on a pavement more than a moving load and may be overly severe for asphaltic materials.

29. In the gyratory testing, the viscous behavior of the bituminous material could be noted in that when the GTM was stopped there was a large drop in the upper roller pressure P_R prior to recording a reading. In the dynamic triaxial tests, the bituminous-stabilized material did perform more favorably with respect to the other materials.

30. The values of the gyratory shear modulus G_G are provided in Tables A5-A9. The gyratory modulus of elasticity E_G is related to the G_G by equation $E_G = 2G_G(1 + \nu)$ where ν is Poisson's ratio. The practical range of Poisson's ratio is between 0 and 0.5, which means E_G will be between $2G_G$ and $3G_G$. From the table, it is seen that the resulting E_G will be very low when compared with the modulus of

elasticity determined from repeated load triaxial tests. It is noted that the G_G's for lower shear angles are greater than for the higher shear angles. Thus, one possible cause for the relatively low values of G_G could be the large magnitudes of shear.

31. Little was gained in the attempts at conducting gyratory tests for materials of varying water content. Either the gyratory shear parameters of the materials were unaffected by changes in moisture content, or the procedures and/or equipment used for retaining the moisture in the samples prevented isolation of the moisture effects. In studies reported by Parker[10] and by Wahls,[11] the effects of moisture content on the strength of fine-grained soils were clearly evidenced by the results of the gyratory shear tests. Since there is no reason to suspect the test procedures or equipment, it is concluded that the changes in moisture content have little influence on the strength of the test materials.

Triaxial Compression

32. One triaxial test was conducted for each of the materials except the base 1A. The description of the tests and the results from the test are contained in Appendix B. A summary of the test results is contained in Table B4. It is immediately apparent that the bituminous-stabilized materials fared much better in the triaxial testing than in the gyratory shear test. The resilient modulus M_R was much higher for the bituminous-stabilized materials than for the nonstabilized materials. Also, in determining the permanent deformation characteristics, a more severe loading of the stabilized materials was required to obtain measurable permanent strain. Even with more severe loading permanent deformation for the BSBC was much less than that of the nonstabilized materials. The influence of temperature on the behavior of the bituminous materials was not investigated, but certainly the performance of these materials at higher temperatures would not have been nearly so impressive.

33. In comparing material I (BSBC) with material III (BSOG), it

was evident that the BSBC was superior to the BSOG. As for the comparison between the two nonstabilized aggregates, the results are not so conclusive. The M_R of material II (NSOG) was higher and the permanent deformation less than for material V (base 5A). At the conclusion of the repeated loading tests, each of the nonstabilized materials was loaded to failure in the manner of a standard triaxial test. In this test, material V (base 5A) had the higher shear strength. Undoubtedly, the effect of cohesion was being reflected in the results of the test. The material II (NSOG) was obviously a cohesionless material, and thus a Mohr's diagram (Figure 1) could be constructed from the single test.

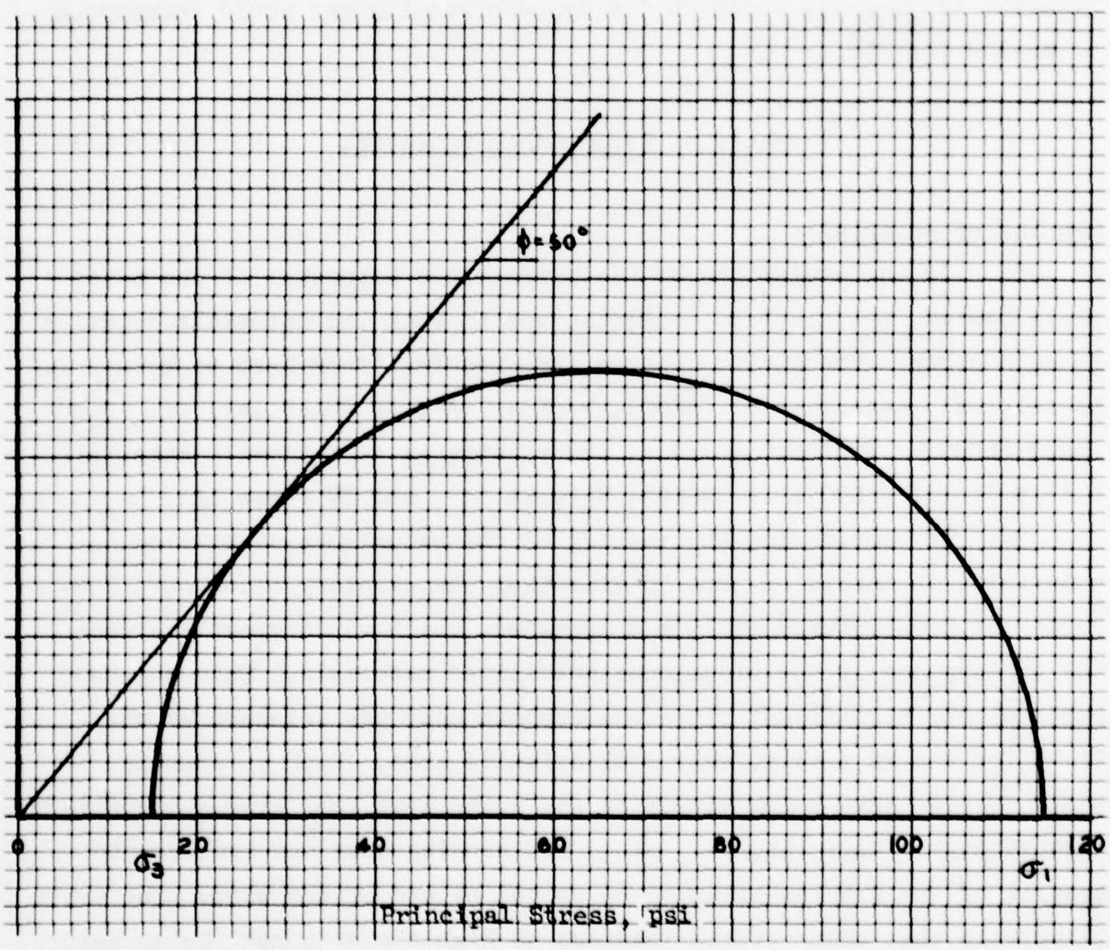

Figure 1. Mohr diagram for open-graded aggregate

This diagram yielded an angle of internal friction ϕ of 50 deg. The material V (base 5A) did possess some amount of cohesion* as evidenced by the manner in which the sample stood without confinement. If the value of cohesion is assumed to be 5 psi for the base 5A, the material would have a ϕ of approximately 48 deg. It can be noted that the ϕ values obtained from these tests agree with the ϕ_G obtained in the gyratory testing (Table 2).

34. The relative ranking as a result of the triaxial testing is as follows: materials I (BSBC), III (BSOG), II (NSOG), and V (base 5A).

35. No triaxial test was conducted on material IV (base 1A), but based on the comparison of the gyratory test, it was felt that this material would have been ranked last. The ranking provided with the triaxial testing was based on the dynamic loading and would correspond to moving traffic.

Material Performance Potential

Ranking of materials

36. In the evaluation of the performance potential of the materials tested, the simplest procedure was to rank the materials relative to each other. For static loading, the ranking (from best to worst) is as follows: materials II (NSOG), V (base 5A), IV (base 1A), III (BSOG), and I (BSBC). For moving loads, the ranking is as follows: materials I (BSBC), III (BSOG), II (NSOG), V (base 5A), and IV (base 1A).

37. In considering the ranking of the bituminous-stabilized materials, it should be kept in mind that the gyratory tests were conducted at temperatures of 75°F or greater, which would greatly affect the performance of the bituminous-stabilized materials. This may be particularly significant since the asphalt used was selected by the NJDOT for use in a relatively cool climatic area. Such an asphalt would tend to have a low viscosity and would have poor strength qualities at higher temperatures. Also, the ranking does not include the benefit of the waterproofing to be gained by use of the BSBC. The tests illustrate that under static loadings at high temperatures the asphalt

* This cohesion would appear to be due to the presence of moisture in the sample.

acts as a lubricant and reduces the shear strength of the aggregate.

38. The ranking of the nonstabilized aggregate indicates the NSOG material to be superior to the base 5A. Again, as with the stabilized materials, the ranking must be considered with regard to the test conditions. In both the gyratory and the triaxial tests, the materials had positive confinement. Such confinement is essential to development of the strength of the NSOG material, whereas the base 5A did possess some cohesion. Also, certain aggregate materials, when compacted in a dense state, tend to develop a cementation and thus an ability to sustain tensile stresses.

39. From the laboratory tests, it appeared that the relatively high percent of fines in base 5A was detrimental to the material's performance. This behavior had been noted in previous studies. A particular study clearly illustrating the effects of fine content was reported by Ferguson.[5] The results of the WES tests agree with the results of Ferguson's tests in that the materials with the lower fine content had higher M_R and lower permanent deformation under repetitive loadings. Agreement of results between gyratory shear testing and triaxial testing was encouraging; that is, the materials with the higher fines content appeared to be the poorer quality materials.

40. For evaluating material IV (base 1A), there are only the results of gyratory shear testing. The results of these tests indicate that of the three nonstabilized aggregate materials the base 1A was the poorest quality. This was as expected since the NSOG and base 5A are both crushed materials having very angular particles, whereas the base 1A was a pit-run material having rounded particles.

41. The conventional bases, i.e., the BSBC, the base 1A, and the base 5A, have an experience data base from which to predict performance and to select placement depths. Thus, the relative ranking of the open-graded materials with respect to conventional bases can be used to determine placement of the open-graded materials. Either of the open-graded materials could replace the base 1A with no anticipated problems. For highways subjected to high-speed traffic, neither of the open-graded materials would match the structural performance of the

BSBC; however, if adequate confinement were provided, both materials would have better performance than the base 5A. One aspect of the better performance of the open-graded materials over the base 5A is that the higher M_R of these materials would result in less fatigue damage to asphalt surfacing. The use of the open-graded materials in place of the base 5A was predicated on the assumption that the material would be adequately confined to prevent intolerable plastic yielding.

Stress resistance

42. The approach for pavement analysis advocated by McRae[9] utilizes the stresses as determined by the Boussinesq stress equations. In the procedure, the shear stress at a point is compared with gyratory shear strength for the vertical stress corresponding to the vertical stress in the pavement. Application of the procedure as given in Reference 9 was attempted; however, for the cases in this study, a complete state of stress and a failure theory were necessary.

43. Consider a pavement subjected to a dual-wheel loading of 5000 lb per wheel at a tire pressure of 78.6 psi. The Boussinesq stresses can be determined by assuming the pavement to be a single-layer system and using a layered-elastic computer program to compute the stresses. The particular program used for this study was the BISAR program developed by Shell Oil Co. The advantages of using the program are that the complete state of stress is computed and computations are for both wheel loadings. For the 6-in. depth, the maximum shear stress is 17.9 psi with a vertical stress of 38.8 psi. For a vertical stress of 38.9 psi, the gyratory strength of the NSOG material would be approximately 50 psi, indicating a factor of safety close to 2.8. However, if the principal stresses (a major principal stress σ_1 of 39-psi compression and a minor principal stress σ_3 of 3.1-psi compression) are considered (Figure 2) with Mohr-Coulomb failure criteria, the indication is that the material fails. For the material to remain in static equilibrium would require the development of additional confining stresses. The overburden does provide some confinement, but for the 6-in. depth this would only be about 0.5 psi. Additional confining stresses can be developed by the passive resistance of the

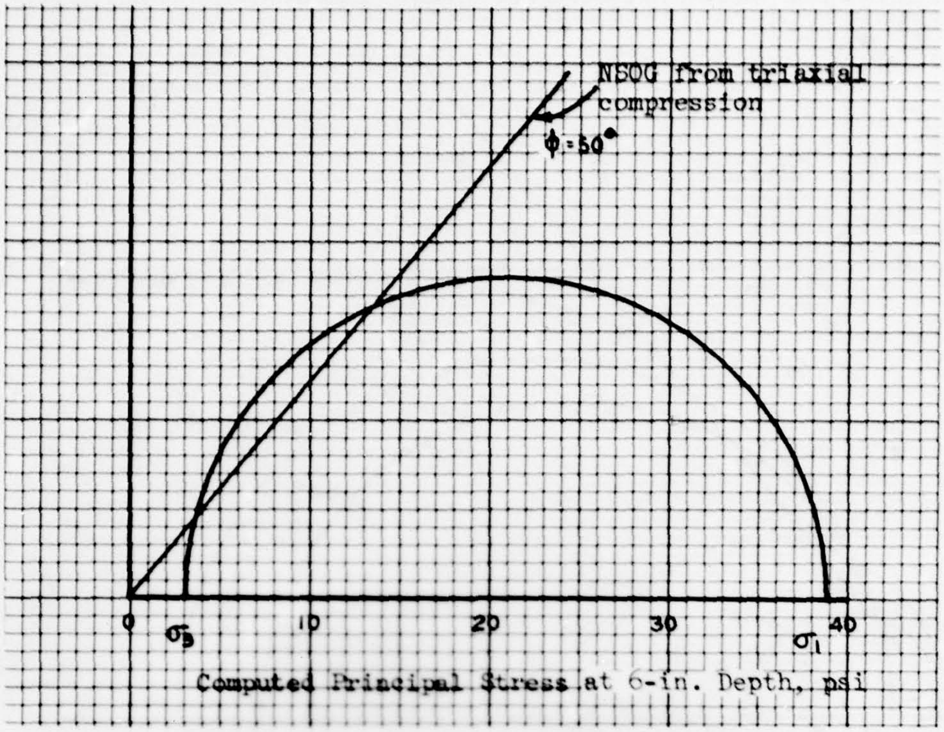

Figure 2. Mohr-Coulomb failure criteria

soil. Estimating from the diagram that a maximum of 2.5 psi additional confinement is needed to maintain stability, the required coefficient of passive earth pressure K_p is 5.0. This value appears quite reasonable and may be a good value with which to consider the other depths. Similarly, Mohr's diagrams were constructed for various depths (Figure 3), and a diagram for the development of confining stresses is shown in Figure 4. The diagrams show that the minimum placement of the NSOG material would be 6 in. The data are not available for analysis, but under moving loads the test results indicate the BSOG material would be placed at depths slightly less than the NSOG. Considering the Barenberg's results with the model test sections, the BSOG probably should be regarded as a cohesionless material and no benefit given to the addition of asphalt. Such a practice would also guard against the possibility of stripping of the asphalt. Thus, the BSOG material would be placed at the same depth as the NSOG material.

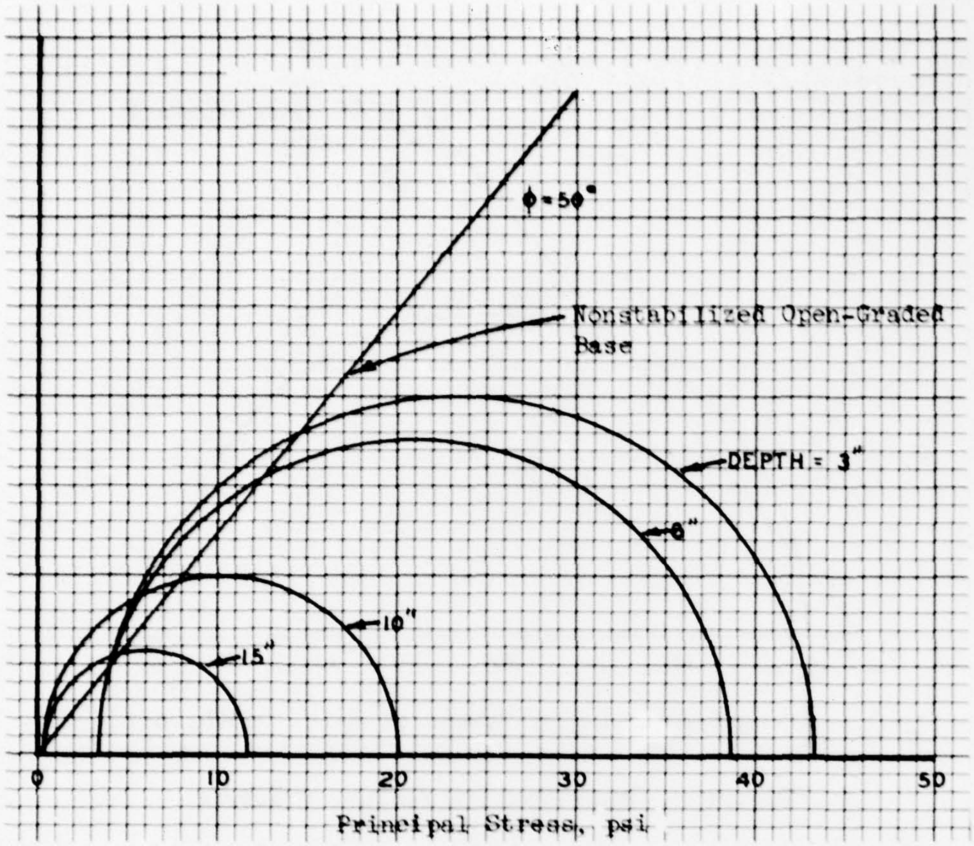

Figure 3. Mohr diagram for different depths of a pavement system with single homogeneous layer

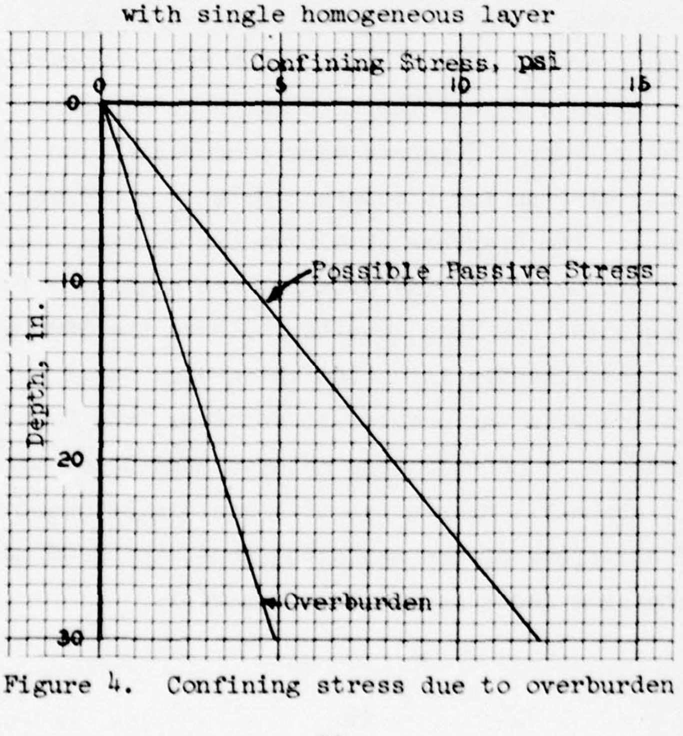

Figure 4. Confining stress due to overburden

44. In the above analysis, there are two assumptions that warrant comment. The first is that a single homogeneous layer was assumed in computing the stresses. For a pavement system this assumption is not true, particularly in the case of a PCC pavement, and would result in computing unrealistically high stresses. For flexible pavements, the assumption is not a serious factor when computing the stress for a slow-moving load during a period of time when the pavement temperature is high. Even for this case, it is felt this stress analysis would lead to conservative stresses. The other assumption is that yielding must occur to develop the passive stresses, and it is assumed that the yielding necessary to develop the confining stresses would be of sufficiently small magnitude so as not to cause cracking of pavement surfacing. In a finite analysis of an airfield pavement, Barker[17] has shown that plastic yielding with the resulting development of passive stresses explains the behavior of granular material. The deformations and strains resulting from the yielding were surprisingly small.

<u>Bearing capacity</u>

45. Another approach, also dependent on passive pressure, is to analyze the pavement based on the bearing capacity formula given by Terzaghi and Peck.[18] The formula for a circular footing is as follows:

$$q = 1.2\, cN_c + \gamma\, D_f\, N_q + 0.6\, \gamma\, r\, N_\gamma$$

where

q = bearing capacity per unit of area

c = cohesion

γ = unit weight of soil

D_f = depth of footing

N_c , N_q , N_γ = bearing capacity factors

r = radius of the footing

46. For checking the bearing capacity of NSOG material placed at the 6-in. depth, it is assumed that the 10,000-lb dual-wheel load is applied to the material by a circular footing having a radius of 6.4 in. Using a ϕ of 50 deg for the material, N_q and N_γ are estimated to

be 320 and 400, respectively. Since the cohesion is zero, there is no need to determine N_c . Using the stated formula, the bearing capacity is computed to be approximately 290 psi. This value is well above the applied stress assumed for the contact pressure of the tires. The analysis has several assumptions that should be considered. First, the bearing capacity formula is for a single static loading to ultimate failure. For repetitive loading where fatigue of the surfacing and cumulative deformation are major considerations, then the allowable load must be lower than the ultimate failure load. Second, except for unit weight, the overburden is assumed to have the same properties as the NSOG material. In actual practice, the material above the NSOG material probably will be of better quality. Third, the radius of the loaded area is computed as if no load distribution were occurring in the material above the NSOG. If the surface material is PCC or a high-grade bituminous concrete, the effective radius of loaded area will be greater. Even with the gross simplifications, the procedures do provide some assurances that large plastic deformations will not occur within NSOG material that is placed with 6 in. of overburden. Fourth, the assumption is made that the thickness of the NSOG material will be sufficient to protect the subgrade and that the subgrade will not affect the strength of the NSOG layer.

47. If the same computations are made for a material such as the base 1A for which a ϕ of 40 deg might be representative, the bearing capacity is only 65 psi. This bearing capacity may be too low, and the ensuing plastic deformations result in an early pavement failure.

48. The two examples serve to illustrate the sensitivity of the bearing capacity to the strength parameter ϕ and give some justification to using a material characterization procedure that provides an indication of the material strength as well as the stiffness.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

49. Based on the laboratory tests and the literature review conducted in this study, the following conclusions are presented:

- a. When provided with adequate overburden for confinement, both the stabilized and nonstabilized open-graded aggregates will perform as highway bases. For normal highway loading, the minimum overburden would be approximately 6 in. With this overburden, the performance of the open-graded bases should be superior to base 1A and on a par with base 5A.
- b. Gyratory testing provides material properties that can be useful in evaluating the performance potential for pavement materials. The most useful parameters appear to be sample density and gyratory shear; the cohesion and shear modulus appear to have limited value.
- c. When gyratory shear tests are conducted on asphalt materials, attention needs to be given to simulation of the rate of loading to which the material is to be subjected in the actual pavement system. Such attention may require development of equipment for measuring the upper roller pressure in a dynamic mode.
- d. Methodology does not exist for adequately quantifying the performance potential for granular materials in pavement systems. Present methodologies rely almost entirely on the resilient properties of material without regard to material strength parameters; although, in the case of the cohesionless materials it was shown that the development of strength to prevent plastic yielding was the prime consideration.

Recommendations

50. The study has justified the following recommendations:

- a. If the open-graded bases are to be used in actual highway pavements, a minimum of 6 in. coverage is to be used.
- b. Additional work should be conducted using the gyratory testing machine for evaluation of pavement materials. Particular attention should be given to the dynamic measurement of the strength parameters. Gyratory shear tests

should be conducted on materials of different qualities in order that the results from the tests might be calibrated to performance. Also, strength studies are needed for correlating the gyratory strength parameters with strength parameters determined by more conventional testing.

c. The equipment and procedures for testing saturated material need additional development work.

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Table 1

Data on Material Density

	Dry Density					
	GTM Compaction					
Material	30 Rev. at 25 psi	30 Rev. at 50 psi	30 Rev. at 200 psi	After Testing	Compaction for Triaxial Sample	Impact Compaction 56 Blows 3 Layers
I (BSBC)	--	--	158.5	159.3	150.2	
II (NSOG)	104.6	111.0	--	114.8	109.01*	
III (BSOG)	--	--	127.9	129.6	121.9	
IV (Base 1A)	134.2	138.1	--	139.9	--	136.8†
V (Base 5A)	135.0	143.5	--	147.3	142.26**	

* Compaction water content = 1.0 which may not be optimum.
** Compaction water content = 6.0.
† Compaction water content = 7.0.

Table 2

Summary Table for Results of Gyratory Shear Tests

Sample No.*	Sample Temperature, °F	Angle	Uncorrected Values for Gyratory Cohesion, psi	Uncorrected Values for Gyratory ϕ_G, degrees	Corrected** Values for Gyratory Cohesion, psi	Corrected** Values for Gyratory ϕ_G, degrees
I-1	75	0.3	13	41.7	--	--
		0.7	75	47.2	--	--
		1.1	16	46.4	--	--
I-1	90	0.3	0	48.5	--	--
		0.7	8	48.5	--	--
		1.1	11	48.5	--	--
I-1	110	0.3	12	51.1	-37.5	28.5
		0.7	2	45.6	-40.1	34.2
		1.1	-5	42.0	-19.8	19.6
I-2	75	0.3	100	21.8	--	--
		0.7	3	45.7	--	--
		1.1	1	50.0	--	--
I-2	90	0.3	27	31.0	--	--
		0.7	11	40.4	--	--
		1.1	9	44.7	--	--
I-2	110	0.3	19	37.6	-6.8	11.0
		0.7	3	46.9	-42.1	33.3
		1.1	2	50.2	-19.0	32.0

(Continued)

* Sample I - bituminous-stabilized base course; Sample II - unstabilized open-graded base; Sample III - bituminous-stabilized open-graded base; Sample IV - NJDOT base 1A; Sample V - NJDOT base 5A.

** Corrections made where possible for wall friction and machine zero.

(Sheet 1 of 5)

Table 2 (Continued)

Sample No.	Sample Temperature, °F	Angle	Uncorrected Values for Gyratory: Cohesion, psi	Uncorrected Values for Gyratory: ϕ_G, degrees	Corrected Values for Gyratory: Cohesion, psi	Corrected Values for Gyratory: ϕ_G, degrees
I-3	110	0.3	6	36.9	-25.3	15.1
		0.7	0	47.7	-32.7	31.0
		1.1	-9	51.3	-27.0	32.4
II-1		0.3	16	39.4	6.3	27.6
Dry		0.7	15	51.8	-7.3	42.6
		1.1	12	57.0	-1.0	51.2
II-2		0.3	14	39.7	5.8	34.6
Dry		0.7	13	50.2	-0.8	40.9
		1.1	9	56.5	-3.2	51.9
II-3		0.3	22	40.0	28.0	35.5
Saturated		0.7	20	52.4	16.2	47.1
		1.1	13	57.2	23.6	49.8
II-4		0.3	18	40.0	27.6	32.6
Saturated		0.7	12	50.0	5.8	46.3
		1.1	12	54.3	22.2	43.9
III-1	75	0.3	16	21.8	8.4	19.7
Dry		0.7	11	35.0	3.2	32.4
		1.1	14	39.0	6.2	41.0
III-1	90	0.3	8	18.3	6.6	14.8
Dry		0.7	11	36.1	6.7	28.6
		1.1	7	38.0	8.7	34.9

(Continued)

Table 2 (Continued)

Sample No.	Sample Temperature, °F	Angle	Uncorrected Values for Gyratory Cohesion, psi	Uncorrected Values for Gyratory ϕ_G, degrees	Corrected Values for Gyratory Cohesion, psi	Corrected Values for Gyratory ϕ_G, degrees
III-1	110	0.3	9	20.3	5.7	18.4
Dry		0.7	13	33.0	2.6	31.4
		1.1	9	39.7	12.2	36.8
III-2	75	0.3	9	25.2	-0.8	22.2
Dry		0.7	11	32.6	-3.2	31.2
		1.1	5	43.2	2.8	40.7
III-2	90	0.3	6	25.2	7.8	15.5
Dry		0.7	4	36.5	-4.8	33.5
		1.1	6	41.3	6.0	38.3
III-2	110	0.3	12	17.7	10.2	11.9
Dry		0.7	6	33.0	-4.3	30.3
		1.1	12	36.5	11.0	33.4
III-3	75	0.3	9	32.2	27.6	25.0
Saturated		0.7	9	35.0	19.8	29.2
		1.1	6	40.4	20.5	36.4
III-3	90	0.3	11	27.0	31.8	18.2
Saturated		0.7	7	35.4	24.2	28.5
		1.1	7	42.3	23.0	38.2
III-3	110	0.3	20	23.3	34.4	21.2
Saturated		0.7	15	31.0	21.2	27.9
		1.1	12	39.0	26.0	35.2

(Continued)

Table 2 (Continued)

			Uncorrected Values for Gyratory		Corrected Values for Gyratory	
Sample No.	Sample Temperature, °F	Angle	Cohesion, psi	ϕ_G, degrees	Cohesion, psi	ϕ_G, degrees
III-4	75	0.3	9	36.5	14.2	30.6
Saturated		0.7	3	42.0	14.7	32.3
		1.1	3	47.7	15.5	43.6
III-4	90	0.3	15	30.1	28.6	24.2
Saturated		0.7	8	36.5	15.4	31.3
		1.1	8	45.0	19.7	40.8
III-4	110	0.3	16	33.4	19.0	35.9
Saturated		0.7	12	40.7	33.5	30.4
		1.1	8	45.6	19.0	42.0
IV-1		0.3	10	35.0	8.3	25.2
1A		0.7	10	46.1	2.2	37.3
5% H_2O		1.1	5	51.3	5.5	44.4
IV-2		0.3	23	30.1	16.5	23.4
7% H_2O		0.7	15	44.7	3.2	39.1
		1.1	20	48.0	11.4	46.5
IV-3		0.3	13	35.4	11.2	25.1
8.8% H_2O		0.7	11	46.9	1.5	41.5
		1.1	18	50.0	15.1	44.8
IV-4		0.3	17	28.8	28.0	23.2
8.8% H_2O		0.7	15	39.4	19.6	31.6
		1.1	15	50.0	21.4	42.6

(Continued)

Table 2 (Concluded)

Sample No.	Sample Temperature, °F	Angle	Uncorrected Values for Gyratory: Cohesion, psi	Uncorrected Values for Gyratory: ϕ_G, degrees	Corrected Values for Gyratory: Cohesion, psi	Corrected Values for Gyratory: ϕ_G, degrees
IV-5		0.3	14	33.4	11.1	21.4
2% H_2O		0.7	10	38.0	2.1	30.1
		1.1	6	45.8	7.0	40.2
V-1		0.3	12	35.0	5.5	22.8
5A		0.7	14	41.3	-0.8	35.5
4% H_2O		1.1	7	48.7	4.3	41.6
V-2		0.3	10	36.9	-0.4	29.5
6% H_2O		0.7	10	47.5	-4.6	41.1
		1.1	20	49.2	11.4	44.0
V-3		0.3	19	33.8	11.0	26.2
6% H_2O		0.7	13	42.3	-0.2	32.3
		1.1	10	51.3	2.2	43.8
V-4		0.3	16	31.4	29.3	21.4
6% H_2O		0.7	5	46.7	3.9	39.9
		1.1	9	50.9	10.5	46.9
V-5		0.3	7	38.7	13.6	29.4
6% H_2O		0.7	7	43.5	13.4	33.6
		1.1	9	47.2	18.6	37.4

(Sheet 5 of 5)

APPENDIX A: GYRATORY SHEAR TESTS

Introduction

1. A laboratory testing program was conducted to determine the various gyratory shear properties of five base materials. These base materials were supplied by NJDOT. The procedures used in processing, compaction, and shear testing of these base materials are described in the following paragraphs.

Material Identification and Preparation

2. Base materials used in the gyratory shear testing program are identified as follows:

- a. Material I - bituminous-stabilized base course (BSBC).
- b. Material II - nonstabilized open-graded (NSOG).
- c. Material III - bituminous-stabilized open-graded (BSOG).
- d. Material IV - "1A" bank-run base (base 1A).
- e. Material V - "5A" crushed stone base (base 5A).

The sources of the materials are listed as follows:

- a. Materials I, II, and III - Kingston trap rock.
- b. Material IV - Ogdensburg quartz, quartzite glacial till.
- c. Material V - Pennington trap rock (crushed).
- d. AC 20, Arco Refinery, Philadelphia, Pennsylvania.
- e. AC 20, Arco, Gloucester.

3. Gradation tests were conducted on four of the five base materials as received from the NJDOT. A gradation test was not run on material I because it had been separated into four sizes prior to shipment to WES. Both the as-received and the NJDOT gradations are shown in Tables A1 and A2. Material IV did not have the same gradation as reported by the NJDOT; therefore, it was separated into various sizes so that blending of the aggregate would be possible. Material V was processed in the same method as material IV. Since the open-graded aggregate was essentially one size and contained a minimum of fines,

it was not separated and recombined but was split down to the weight required for making gyratory samples. Table A3 gives the gradations of the various aggregates after being processed and recombined or split. These are the control gradations of the aggregate prior to gyratory shear testing. Gradation curves depicting these five materials are shown in Figure A1.

Sample Preparation

4. After the aggregates were processed, the amount required to produce a 6-in.-diameter sample, 3.75 in. high, was determined. Aggregates representing materials II, IV, and V were then mixed with a preselected quantity of water. The aggregate and water were allowed to equilibrate in a sealed container for a minimum of 24 hr before being compacted and tested. The various water contents used in each material are shown in Table A4.

5. The bituminous-stabilized mixtures were prepared by heating the aggregate to 300°F and the asphalt cement to 270°F. Material I aggregates were mixed with 4.8 percent asphalt, and the material III aggregates were mixed with 3 percent asphalt.

Test Equipment

6. The GTM used in this laboratory testing program was a model 6B, serial No. 1, which utilized a 6-in.-diameter sample mold. The GTM was also equipped with an oil-filled roller. Details of the GTM and operational information are described in ASTM D 3387.[19]

7. In an effort to stop the leakage of water from the sample mold during compaction and testing, O-rings were placed at the bottom and top of the test samples. When it became apparent that the use of O-rings alone would not sufficiently stop the leakage of water from the mold whenever saturated samples were being tested, a special base plate was made to prevent leakage. This special base plate was referred to as an O-ring base plate. It was only used when saturated samples were compacted and tested.

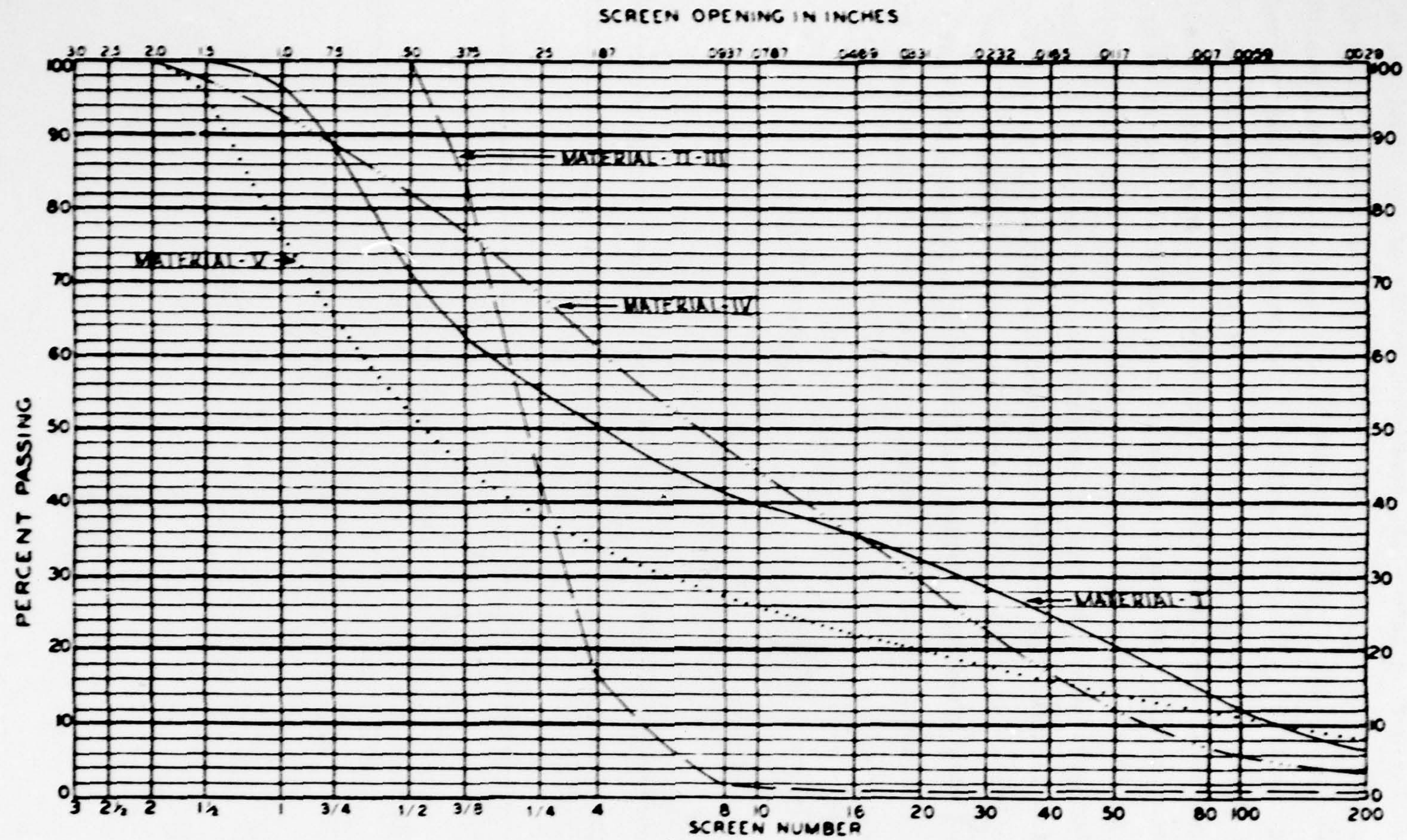

Figure A1. Control gradations prior to gyratory testing, all materials

8. The O-ring base plate consisted mainly of two sections, a bottom plate and a top plate. The bottom plate was drilled and tapped in the center so water could be added to a sample after the compaction had been completed. The top surface of the bottom plate was grooved in a radial design with all the grooves meeting at the center of the plate. These grooves allowed water to be distributed uniformly over the bottom of the test sample. The top section of the base plate had holes drilled through it to allow passage of the saturation water from the grooves of the bottom portion of the base plate into the test sample. Both the top and bottom sections had grooves cut around the periphery so that O-rings could be installed. This O-ring base plate and a similar top plate were also used when applying a vacuum just prior to saturating the test samples. Figure A2 shows the general relationship of the O-rings to the base plate.

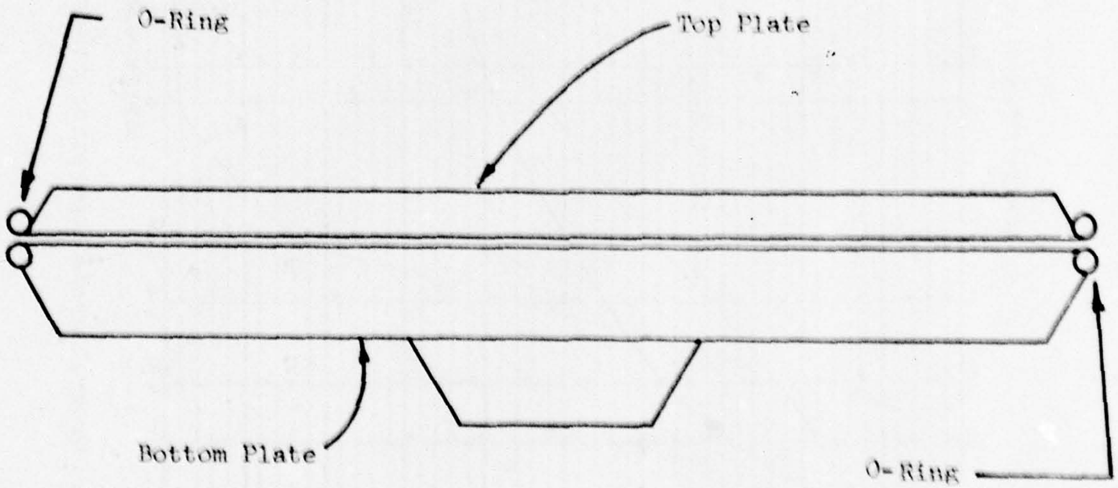

Figure A2. Side view of O-ring base plate

Sample Compaction

9. The compaction basically consisted of applying a specified number of revolutions of the GTM at a specified gyratory angle and vertical pressure to the prepared sample material. The procedures used for materials III, IV, and V are given as follows:

a. Paper disks were placed in the bottom of the mold and on top of the base sample material to prevent the material from adhering to the base plate and top plate of the GTM. Two filter papers, Schleicher and Schnell No. 595, were substituted for each paper disk when compacting a sample that would be saturated prior to shear testing.

b. O-rings were also placed in the bottom of the mold, on top of the paper disk, and on top of the loose sample material. When samples to be saturated were compacted, the bottom base plate of the GTM was replaced with the special O-ring base plate.

c. The GTM gyratory angle was set at 1.1 deg.

d. After the mold and sample material were secured in the GTM, 30 revolutions at a vertical pressure of 25 psi were applied to the sample.

e. After the first 30 revolutions at 25 psi were completed, the height of the sample was recorded before increasing the vertical pressure to 50 psi and applying an additional 30 revolutions of compaction.

f. Most samples were left in the GTM for shear testing except those to be saturated prior to shear testing.

10. The bituminous-stabilized materials (I and III) were not compacted with the same compaction effort used on the nonstabilized materials. Mixtures for materials I and III were compacted at a temperature of 250°F in the GTM with 30 revolutions at a gyratory angle of 1.1 deg and a vertical pressure of 200 psi.

11. The compacted unit weights and water contents for all the test samples are given in Table A4.

12. During the initial phases of compacting materials IV and V and before incorporating the above compaction procedures, two problems were encountered. One problem was obtaining the required unit weights specified by the NJDOT. Only one or two revolutions of the GTM produced a unit weight higher than that specified, and additional revolutions, which

occurred during the shear testing, continued to increase the sample unit weight and affect the shear properties to a large degree. Therefore, after some discussion, it was decided to use a higher but more constant unit weight that would not be affected by the addition of GTM revolutions from the shear testing. To obtain this higher unit weight, 30 revolutions at 25 psi, followed by 30 additional revolutions at 50 psi, were selected to be the compaction effort on materials II, IV, and V.

13. The other problem was the water content of the compacted samples. As samples were compacted in the GTM, water was frequently squeezed from the sample. Therefore, it was not always possible to compact a sample at the optimum water content specified by the NJDOT. Table A4 lists the initial water content of the sample before compaction and the final water content after compaction.

Saturating Samples

14. Of the 21 samples tested in this laboratory program, seven were saturated with water prior to being shear tested in the GTM. These saturated samples consisted of two each from materials II, III, and V and one from material IV. The procedures used in saturating these samples are as follows:

- a. The mold containing the sample and the O-ring base plate were removed from the GTM after compacting the sample.
- b. A small pipe and hose with a control valve were connected to the O-ring base plate. This hose was used to supply water to the sample through the O-ring base plate.
- c. The mold, base plate, and sample were then placed on a stand, and a cover plate was placed over the mold. This cover plate was designed to seal the top of the mold so a vacuum could be produced on the sample. The O-ring base plate provided the seal at the bottom of the sample.
- d. A vacuum line and gage were then connected to the cover plate.
- e. The control valve on the water supply hose was closed before applying the vacuum.
- f. A vacuum of 27 in. of mercury was applied for 1 hr.

g. After 1 hr, the water control valve was opened and water was allowed to enter the sample from the bottom. When free water appeared on the top surface of the sample, the water control valve was closed, and the sample was allowed to soak for 1 hr prior to shear testing.

A general cross-sectional view of the device used to saturate the samples is shown in Figure A3.

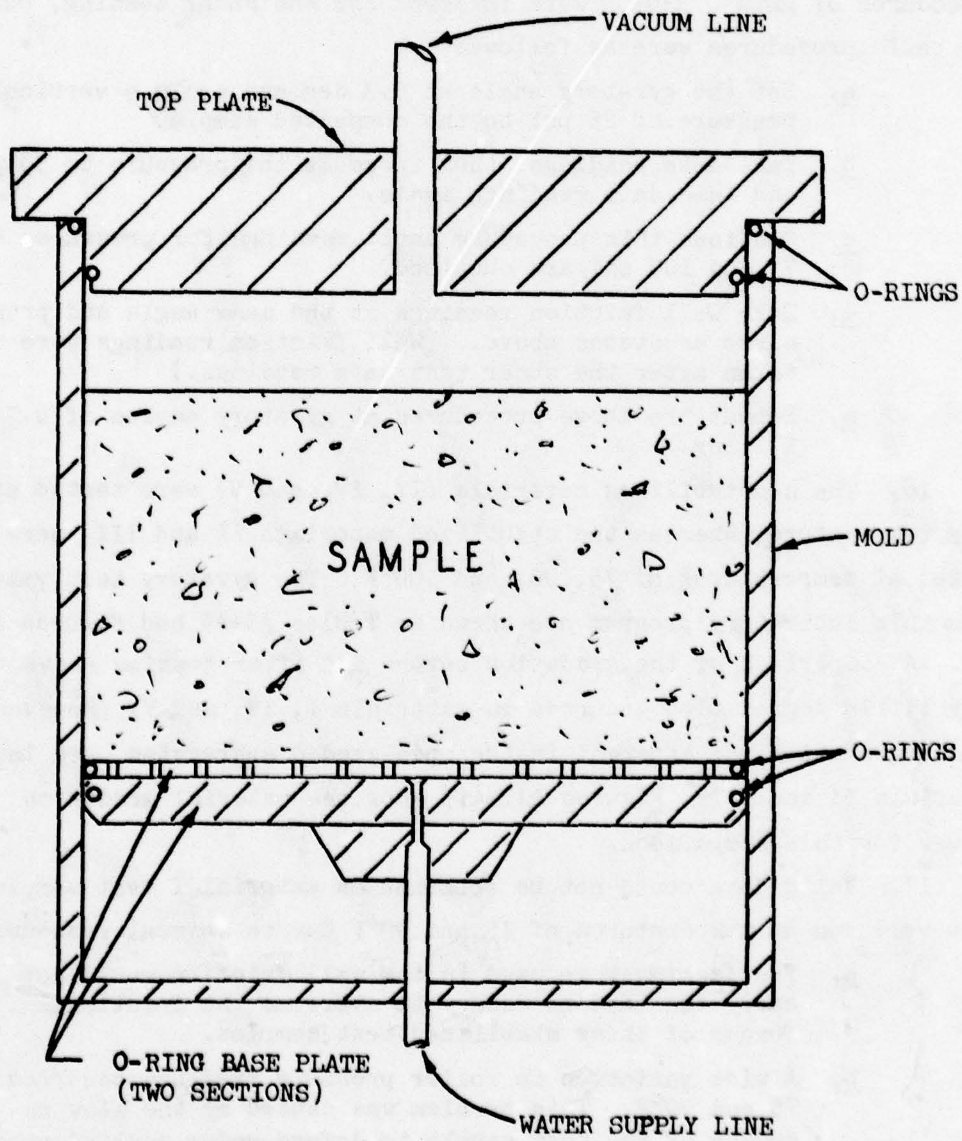

Figure A3. Cross-sectional view of vacuum saturation device

Testing

15. Each sample was shear tested at three different gyratory angles and four vertical pressures in accordance with ASTM D 3387.[19] The three gyratory angles were 0.3, 0.7, and 1.1 deg, and the four vertical pressures were 25, 50, 75, and 100 psi. The detailed testing procedures of ASTM D 3387[19] were followed for the shear testing, but the basic procedures were as follows:

- a. Set the gyratory angle at 0.3 deg and apply a vertical pressure of 25 psi to the compacted sample.
- b. Take data readings, then increase the pressure to 50 psi and take data readings again.
- c. Continue this procedure until readings for pressures of 75 and 100 psi are obtained.
- d. Take wall friction readings at the same angle and pressures as stated above. (Wall friction readings were taken after the shear test data readings.)
- e. Repeat the above procedures at gyratory angles of 0.7 and 1.1 deg.

16. The nonstabilized materials (II, IV, and V) were tested at room temperature, whereas the stabilized materials (I and III) were tested at temperatures of 75, 90, and 100°F. The gyratory test values from this laboratory program are shown in Tables A5-A9 and Figures A4-A10. A comparison of the gradation before and after testing shows that very little degradation occurred in materials I, IV, and V. However, some degradation was apparent in the open-graded aggregates used in materials II and III. Figures A11-A15 show the material gradation curves for this comparison.

17. Valid data could not be obtained on material I test samples that were run at temperatures of 75 and 90°F due to several reasons:

- a. The jacking yoke used in the wall friction would not carry the load necessary to overcome the frictional forces of these stabilized test samples.
- b. A wide variation in roller pressure readings occurred at 75 and 90°F. This problem was caused by the slow response of the test sample to deform under applied pressure; therefore, a wide range of roller pressures could be obtained under a given set of loading conditions.

c. This slow response also influenced the sample height readings.

18. Calculations and corrections of the laboratory test data were made in accordance with ASTM D 3387.[19] The GTM corrections used to correct the calculated shear values are shown below for gyratory angles 0.3, 0.7, and 1.1 deg. These GTM corrections are part of the requirements of ASTM D 3387, Annex A2.[19] Data for O-ring base plate corrections are as follows:

	Machine Correction Values, psi		
	0.3	0.7	1.1
With wall friction, readings included	+21.8	+14.0	+19.9
Without wall friction, readings included	+0.3	-9.5	-4.7

Correction values for standard base plate are as follows:

	Machine Correction Values, psi		
	0.3	0.7	1.1
With wall friction, readings included	+2.1	-4.8	+5.2
Without wall friction, readings included	-8.2	-17.2	-10.2

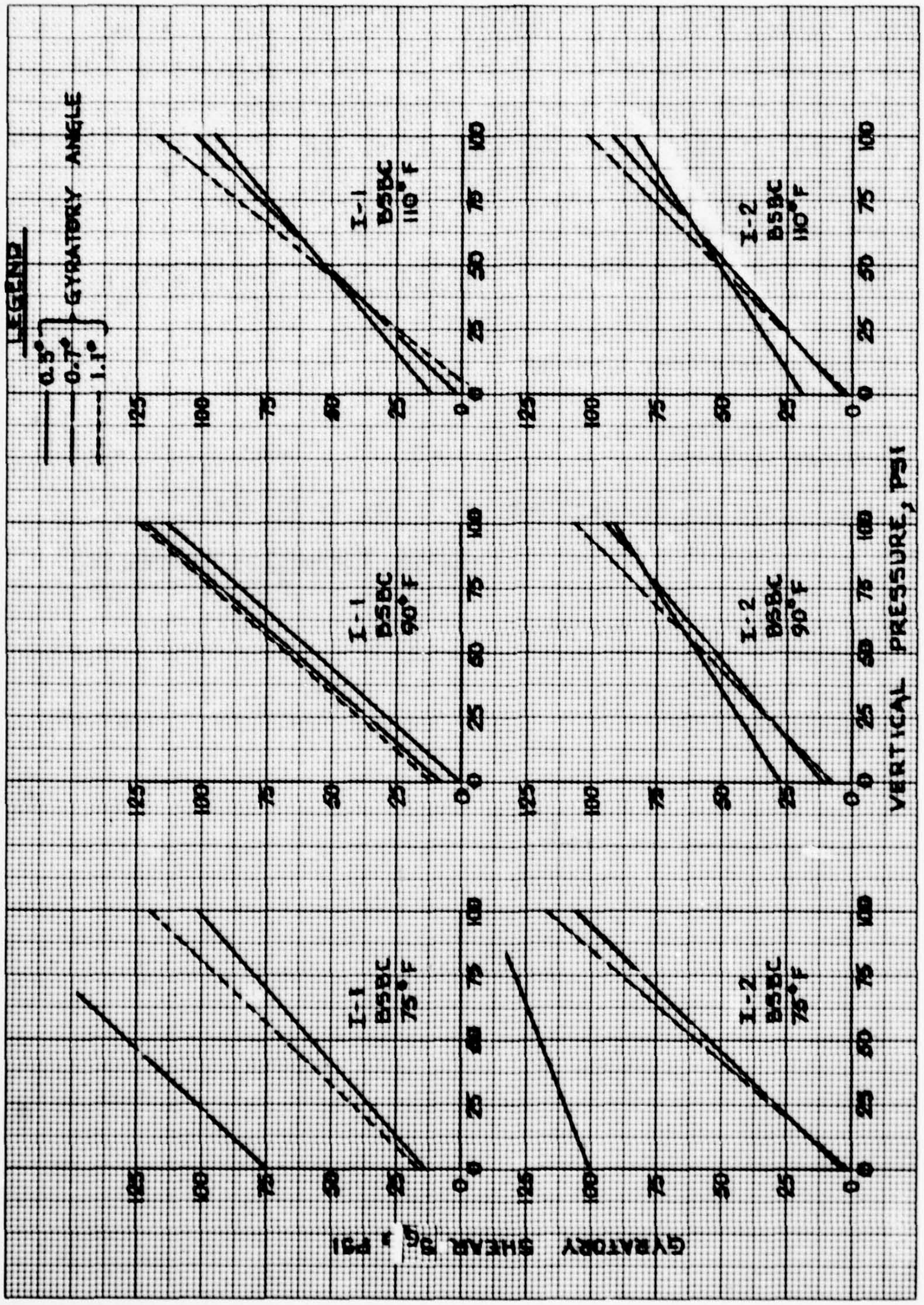

Figure A4. Uncorrected gyratory shear versus vertical pressure, materials I-1 and I-2

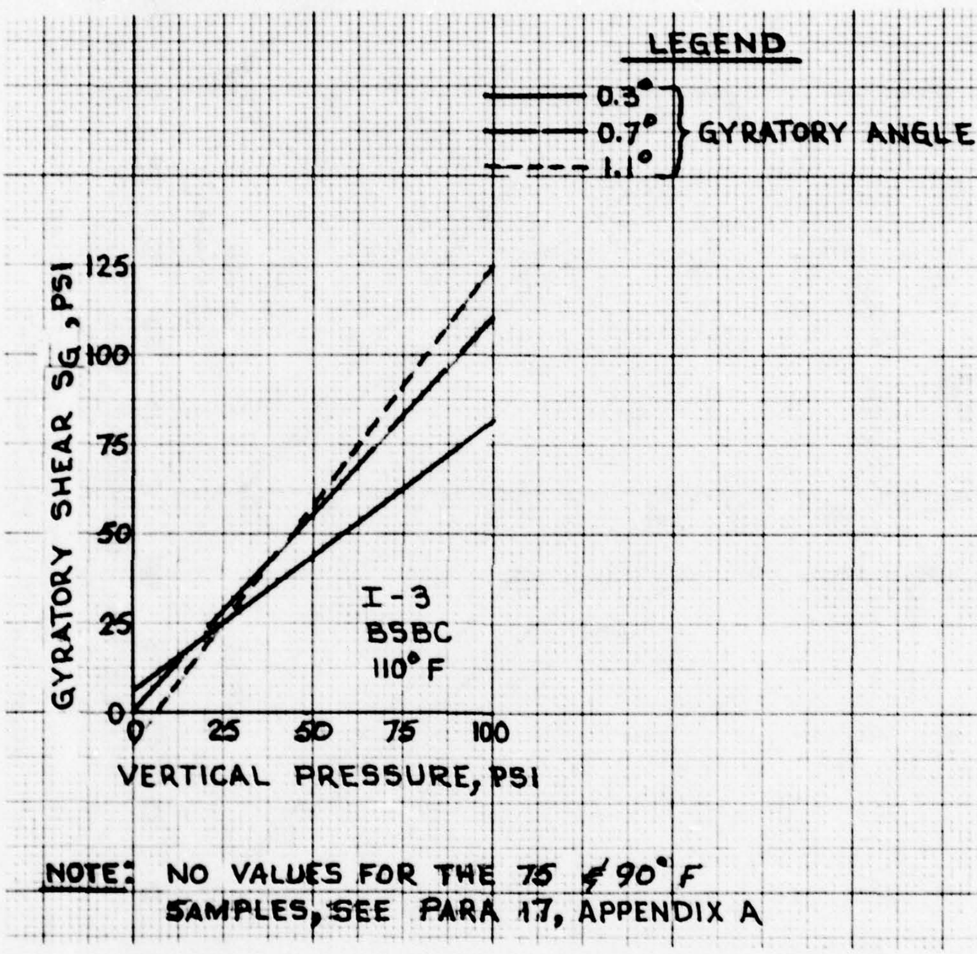

Figure A5. Uncorrected gyratory shear versus vertical pressure, material I-3

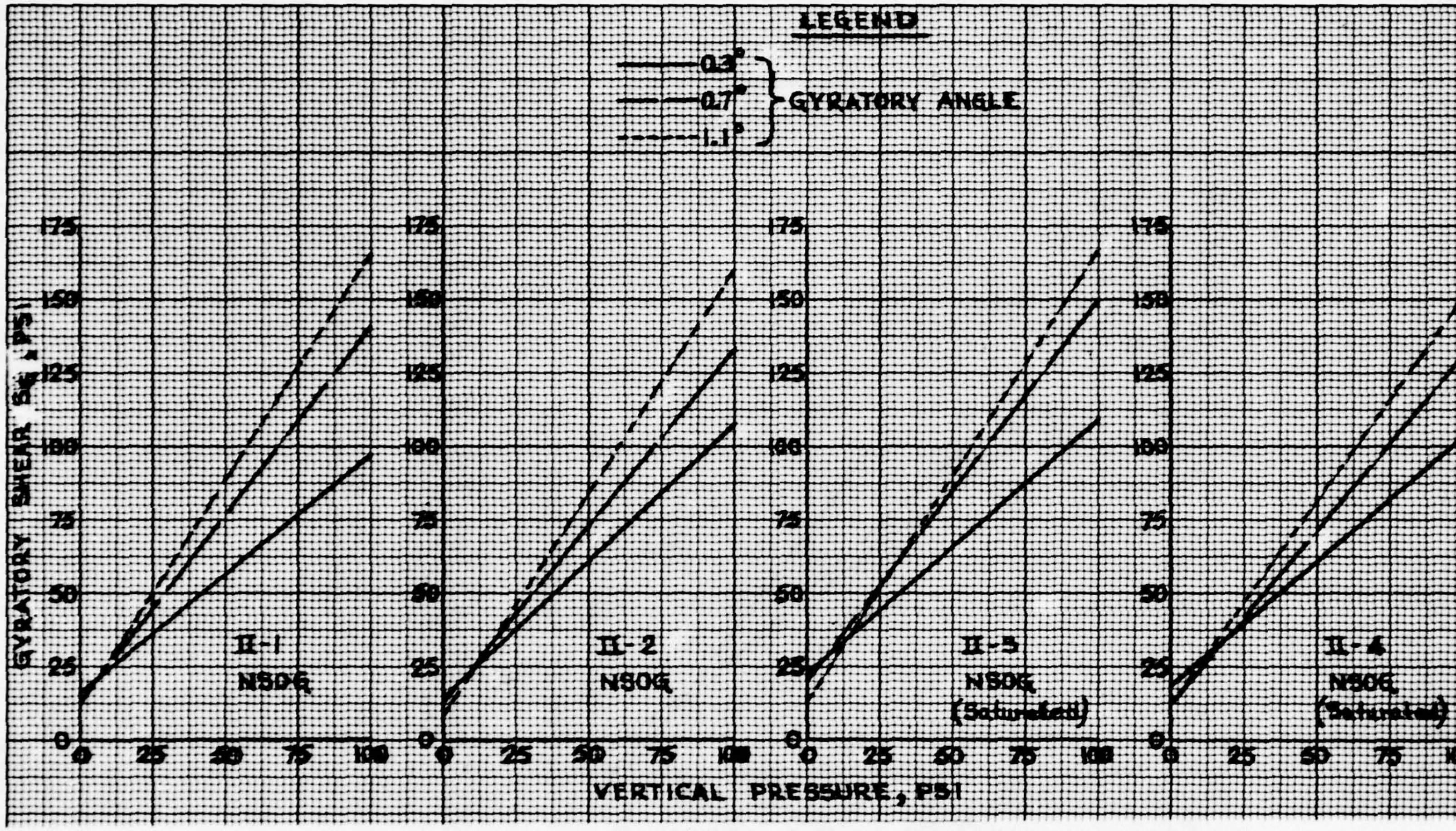

Figure A6. Uncorrected gyratory shear versus vertical pressure, materials II-1, II-2, and II-3

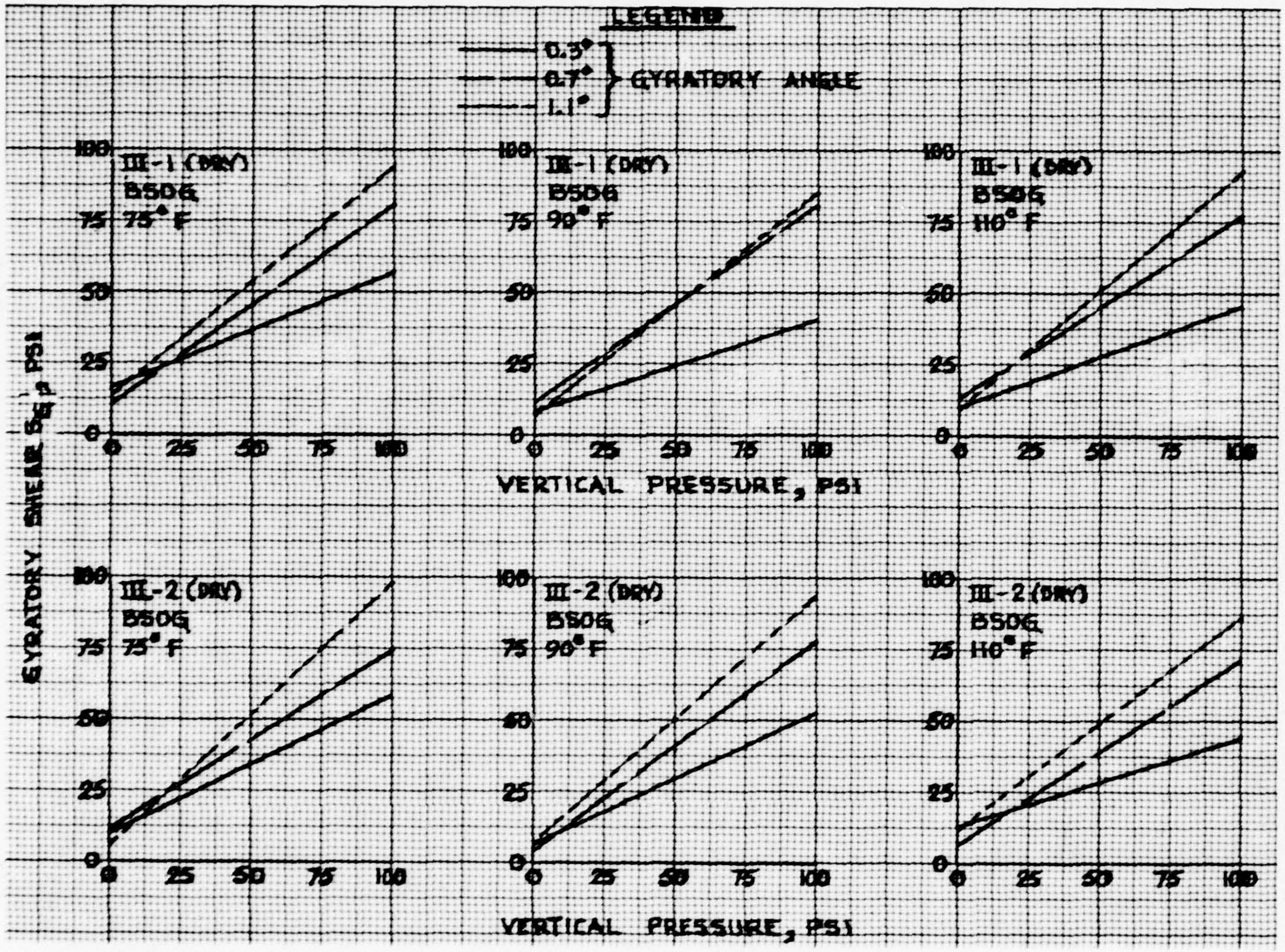

Figure A7. Uncorrected gyratory shear versus vertical pressure, materials III-1 and III-2 (dry)

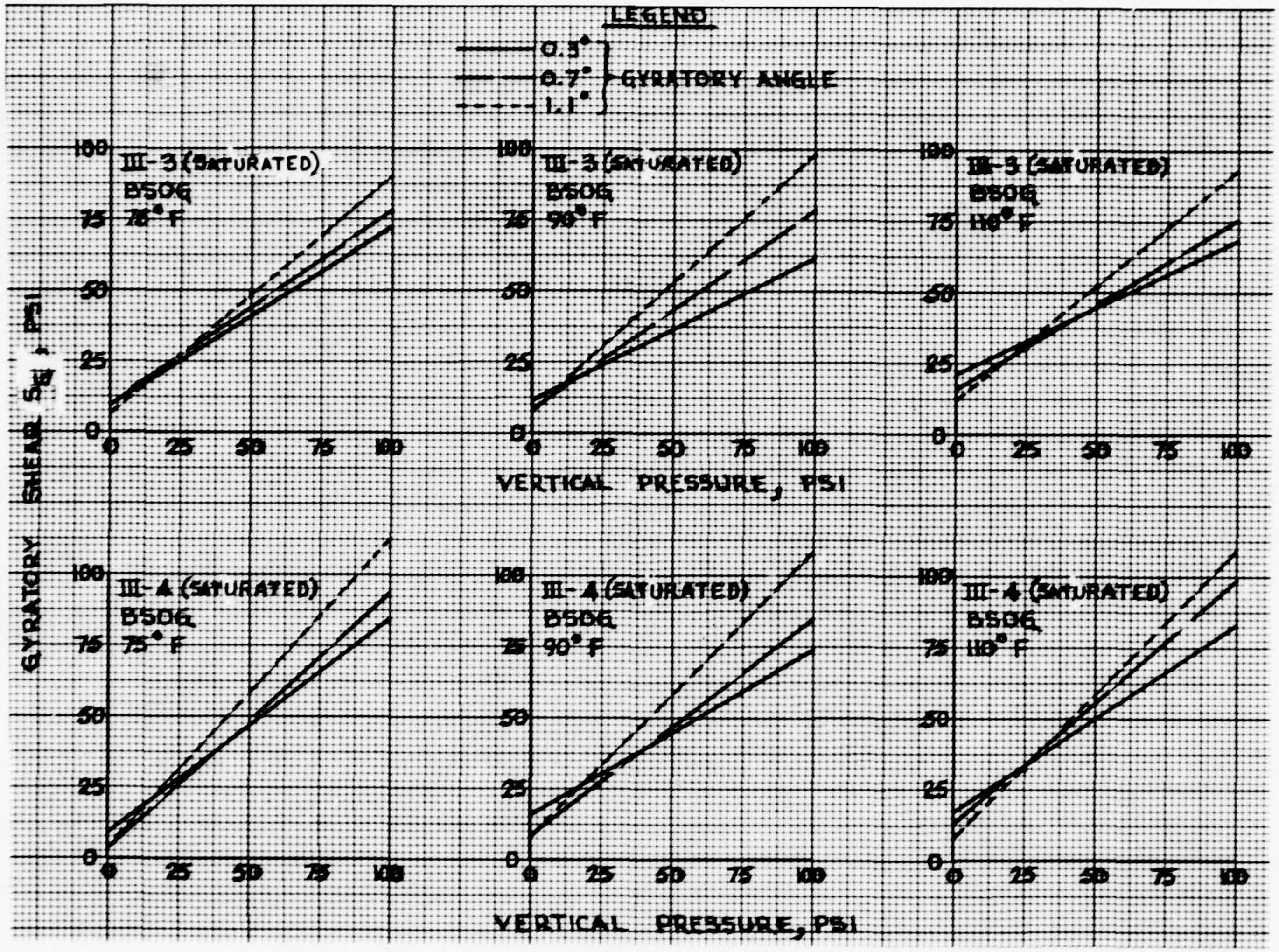

Figure A8. Uncorrected gyratory shear versus vertical pressure, materials III-3 and III-4 (saturated)

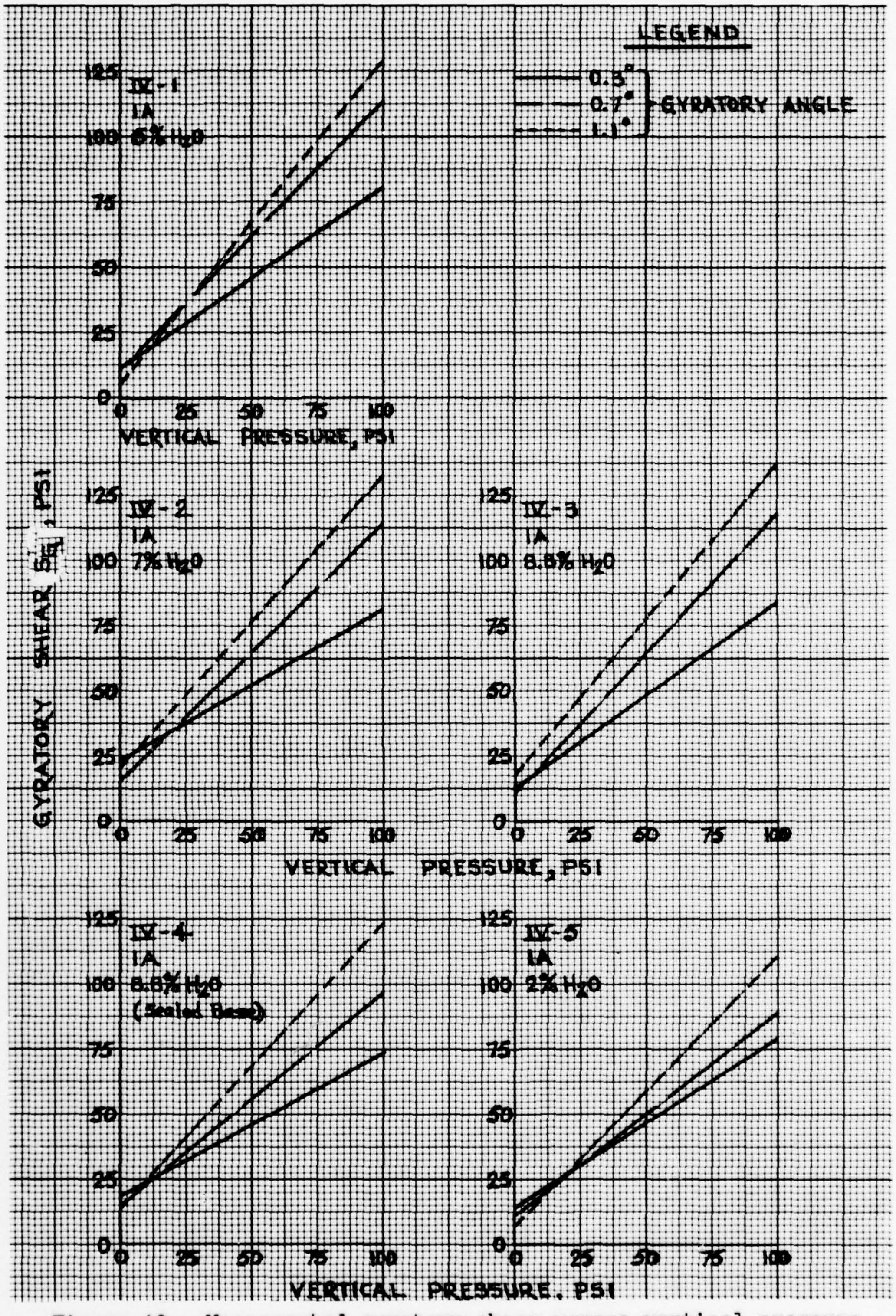

Figure A9. Uncorrected gyratory shear versus vertical pressure, materials IV-1 through IV-5

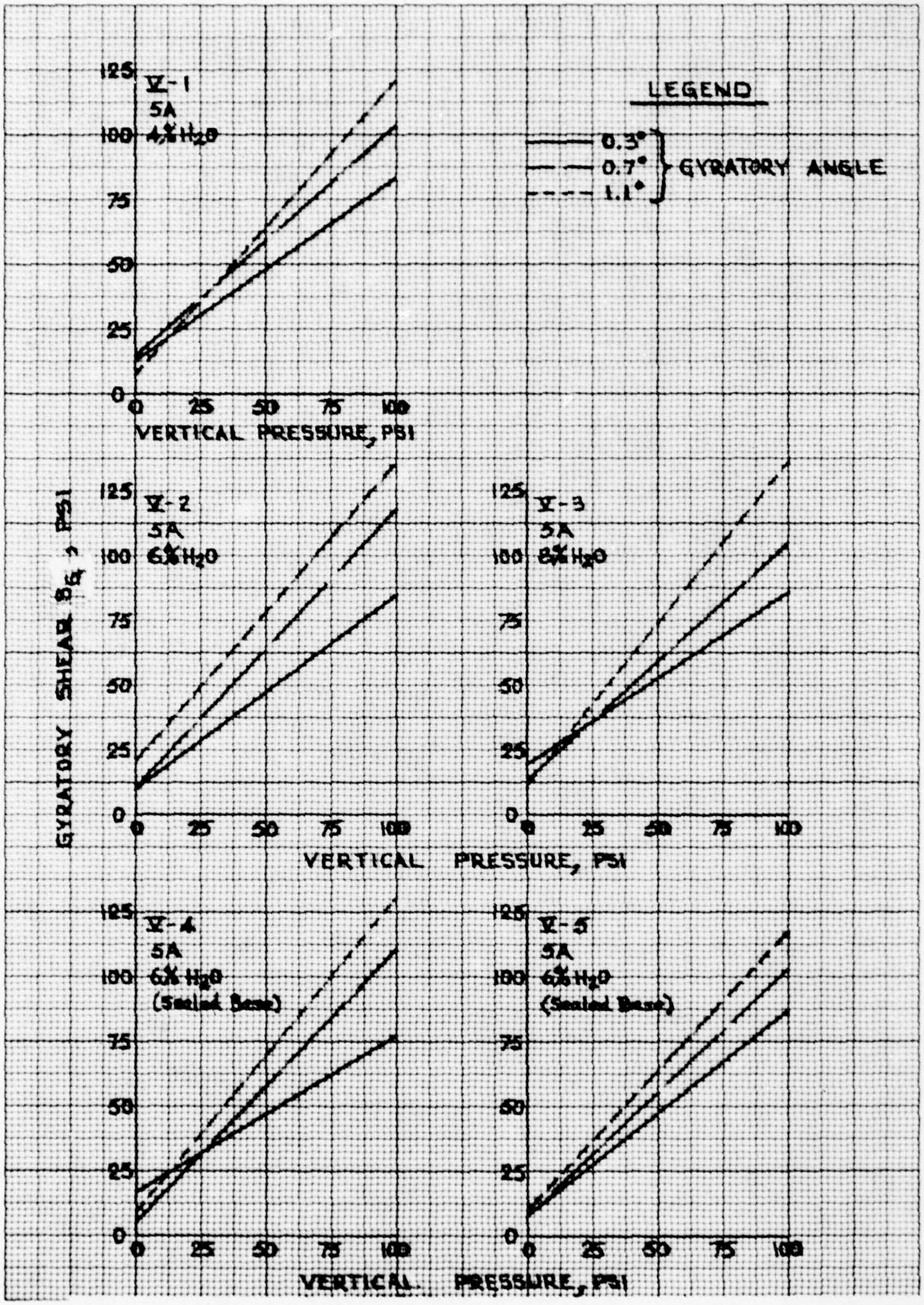

Figure A10. Uncorrected gyratory shear versus vertical pressure, materials V-1 through V-5

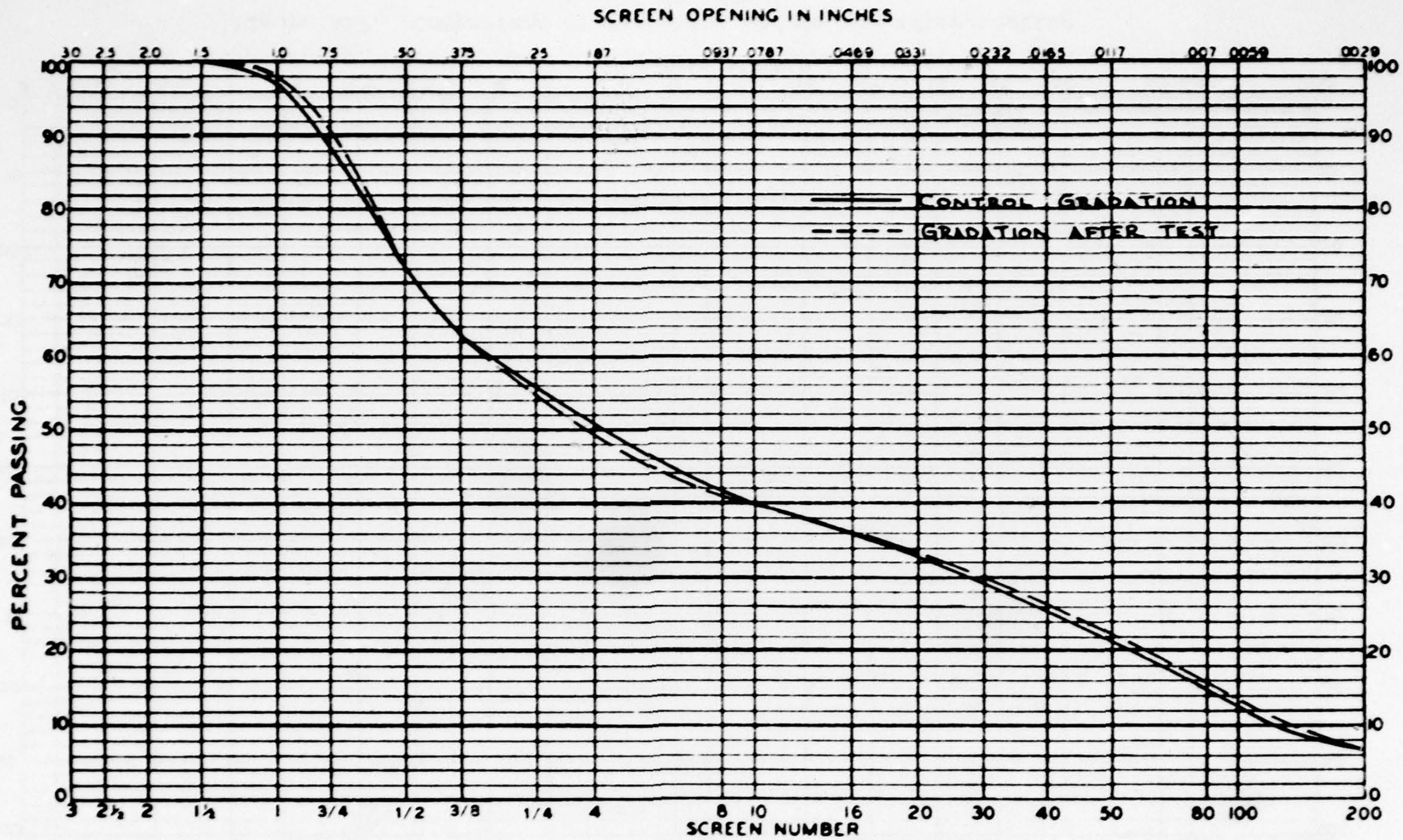

Figure A11. Comparison of gradation before and after testing material I (BSBC)

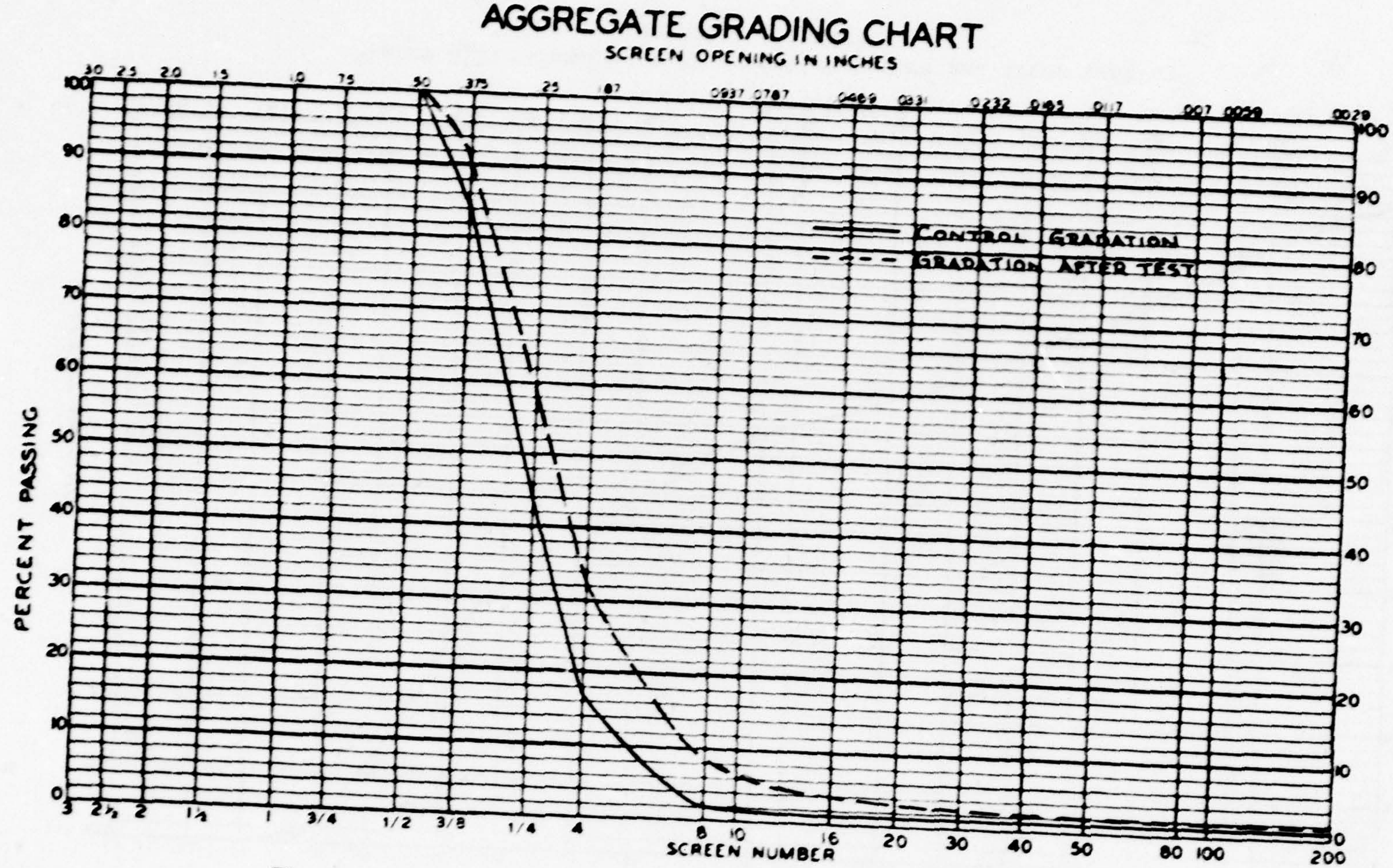

Figure A12. Comparison of gradation before and after testing material II (NSOG)

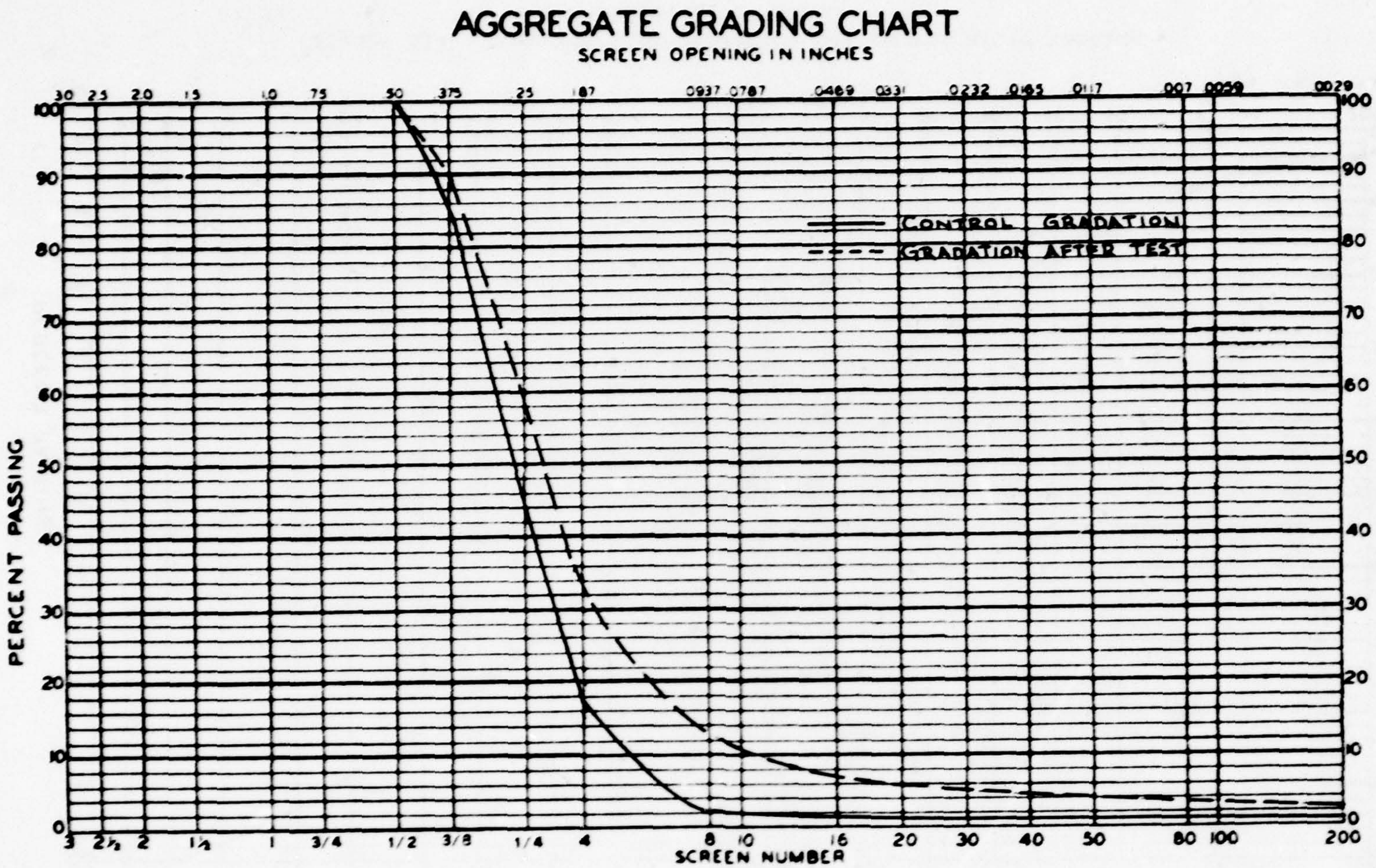

Figure A13. Comparison of gradation before and after testing material III (BSOG)

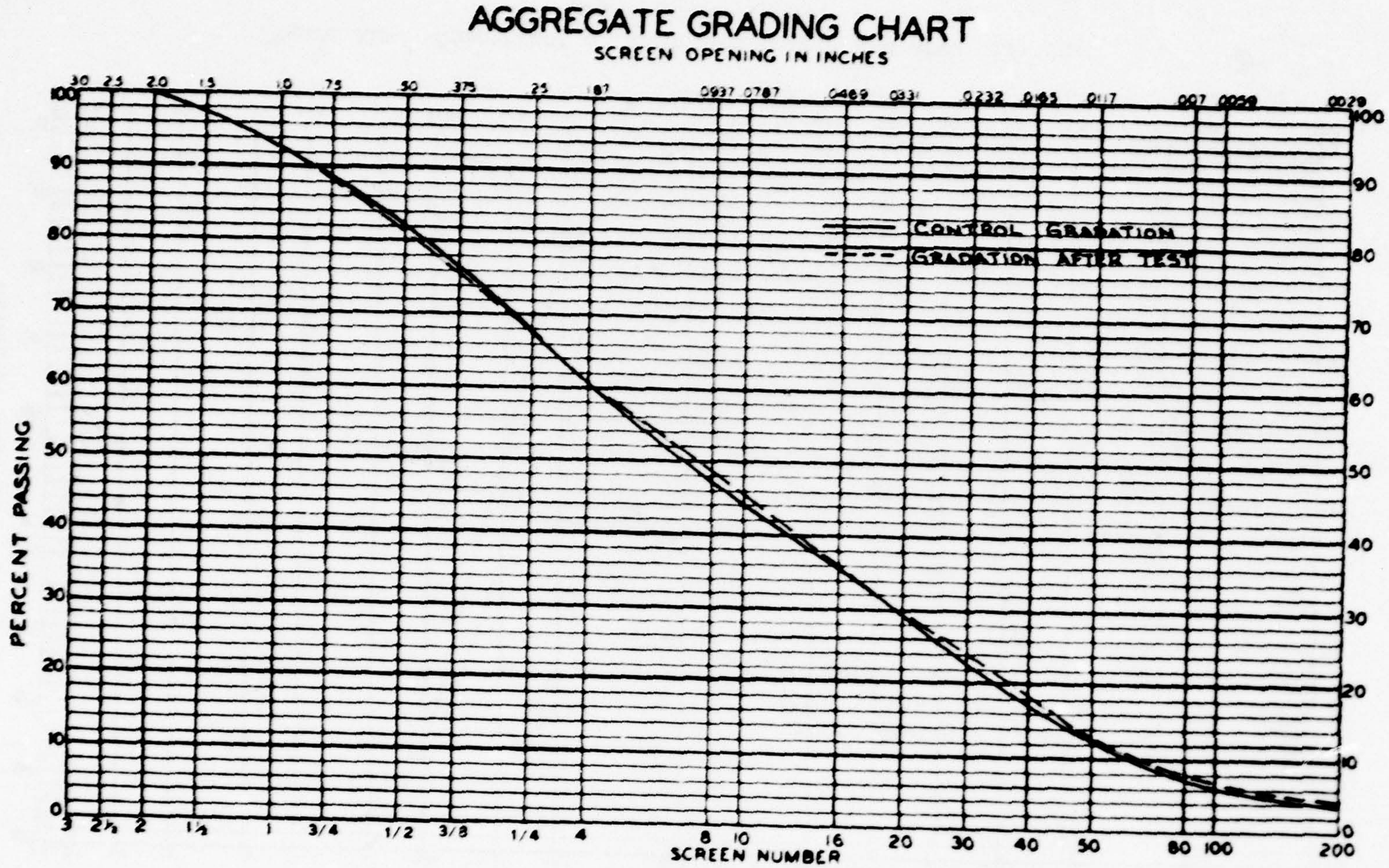

Figure A14. Comparison of gradation before and after testing material IV (1A)

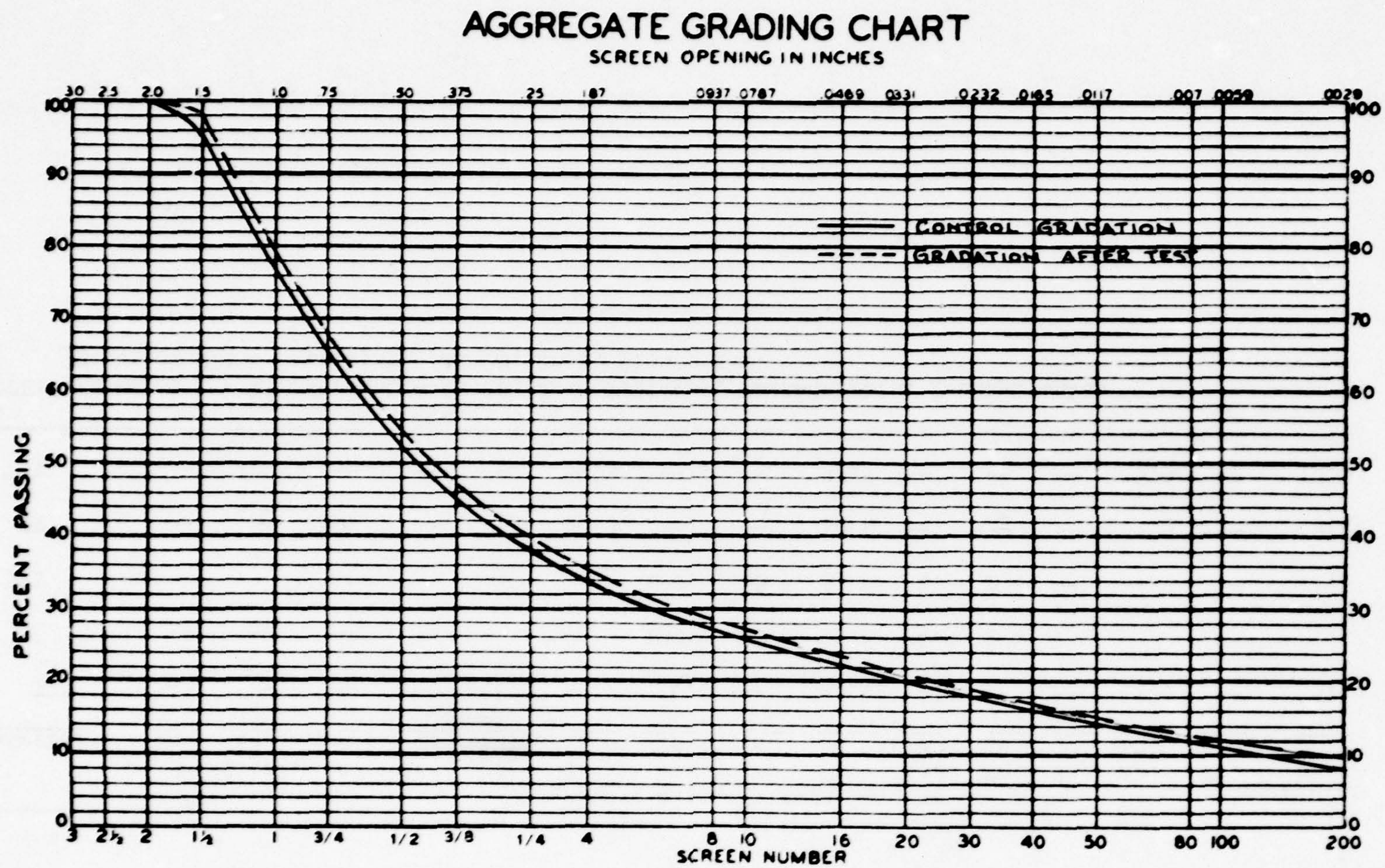

Figure A15. Comparison of gradation before and after testing material V (5A)

Table A1

New Jersey Department of Transportation Aggregate Gradations

Material	Sieve Size, Percent Passing														
	3 in.	2-1/2 in.	2 in.	1-1/2 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
I	--	--	--	100	98.0	--	70.0	--	48.0	42.0	--	--	21.0	--	5.4
II	--	--	--	--	--	--	100	88.5	25.6	4.5	2.4	--	--	--	1.5
III	--	--	--	--	--	--	100	88.5	25.6	4.5	2.4	--	--	--	1.5
IV	--	--	100	--	--	86.3	--	--	58.6	--	--	--	11.3	--	3.2
V	--	100	--	--	--	69.3	--	--	38.5	--	--	--	16.9	--	10.8

NOTE: Material I - 4.8 percent - AC 20, Arco Refinery, Philadelphia, Pennsylvania; Material III - 3 percent - AC 20, Arco, Gloucester.

Table A2

Gradation of As-Received Aggregates

	Sieve Size, Percent Passing														
Material	3 in.	2-1/2 in.	2 in.	1-1/2 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
II	--	--	--	--	--	--	100	86.8	17.3	2.1	1.1	1.1	0.9	0.9	0.7
III	--	--	--	--	--	--	100	86.8	17.3	2.1	1.1	1.1	0.9	0.9	0.7
IV	100	84.6	71.4	71.4	60.6	57.6	49.2	45.8	36.5	28.0	20.0	12.8	6.6	3.4	2.0
V	100	100	100	89.5	70.5	58.4	47.6	42.0	32.1	26.7	22.2	18.2	14.4	11.4	8.7

Table A3

Control Gradations Prior to Gyratory Testing

	Sieve Size, Percent Passing														
Material	3 in.	2-1/2 in.	2 in.	1-1/2 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
I	--	--	--	100	97.1	88.0	72.4	63.3	52.0	42.2	37.2	29.9	22.2	13.1	7.0
II	--	--	--	--	--	--	100	86.8	17.3	2.1	1.1	1.1	0.9	0.9	0.7
III	--	--	--	--	--	--	100	86.8	17.3	2.1	1.1	1.1	0.9	0.9	0.7
IV	--	--	100	--	93.1	88.9	82.4	76.9	61.2	48.3	36.1	23.8	12.5	6.5	4.0
V	--	--	100	89.5	70.5	58.4	47.6	42.0	32.1	26.7	22.2	18.2	14.4	11.4	8.7

Table A4

Gyratory Test Sample Data

Material	Test No.	Water Content, % Initial	Water Content, % Final	Compacted Unit Weights, pcf* 200 psi at 1.1°**	25 psi at 1.1°†	50 psi at 1.1°††
I	1	--	--	159.8	--	--
	2	--	--	158.1	--	--
	3	--	--	157.7	--	--
II	1	--	--	--	103.3	109.6
	2	--	--	--	103.4	109.5
	3	1.0	15.1‡	--	105.7	112.3
	4	1.0	12.6‡	--	106.1	112.4
III	1	--	--	129.6	--	--
	2	--	--	128.9	--	--
	3	--	--	126.2	--	--
	4	--	--	126.9	--	--
IV	1	5.0	5.0	--	132.8	136.8
	2	7.0	5.6	--	134.6	138.9
	3	8.8	5.5	--	135.2	139.2
	4	8.8	9.1‡	--	134.9	138.8
	5	2.1	2.1	--	133.7	137.4
V	1	4.8	4.8	--	127.9	136.4
	2	6.6	6.6	--	136.6	144.7
	3	8.0	7.9	--	138.7	146.9
	4	6.0	9.0‡	--	137.2	145.3
	5	6.0	8.3‡	--	134.8	144.3

* Total unit weights for test samples I and III; dry unit weights for test samples II, IV, and V.

** GTM compaction effort of 30 revolutions at 1.1° gyratory angle and 200-psi vertical pressure.

† GTM compaction effort of 30 revolutions at 1.1° gyratory angle and 25-psi vertical pressure.

†† Vertical pressure was increased to 50 psi, and an additional 30 revolutions of GTM compaction effort was applied to the test sample.

‡ Water content after test sample has been saturated.

Table A5

Gyratory Shear Data--I

Test No.	Test Temp °F	Initial Angle deg θ_o	Vertical Press. psi P_v	Avg Height in.	Avg Roller Press. psi P_R	Uncorrected Shear, psi S_G	Max Gyratory Angle, deg θ_{max}	Measured Max Strain %	Wall Friction Force lb F*	Shear Stress psi**,†	Shear Modulus psi†
I-1	75	0.3	25	3.644	32	33.7	0.36	0.6			
			50	3.636	63	55.4	0.30	0.5			
			75	3.630	101	80.1	0.28	0.5			
			100	3.623	136	99.7	0.25	0.4			
		0.7	25	3.681	74	100.4	1.09	1.9			
			50	3.668	135	159.9	0.95	1.6			
			75	3.646	164	154.3	0.75	1.3			
			100	3.628	170	150.5	0.70	1.2			
		1.1	25	3.676	34	29.9	1.10	1.9			
			50	3.665	87	73.5	1.06	1.8			
			75	3.648	114	95.9	1.05	1.8			
			100	3.633	132	111.8	1.05	1.8			
	90	0.3	25	3.624	27	27.8	0.35	0.6			
			50	3.620	51	52.7	0.35	0.6			
			75	3.616	82	84.7	0.35	0.6			
			100	3.615	110	113.7	0.35	0.6			
		0.7	25	3.637	36	34.1	0.75	1.3			
			50	3.622	70	66.5	0.75	1.3			
			75	3.618	90	88.2	0.77	1.3			
			100	3.617	100	115.2	0.90	1.6			
		1.1	25	3.645	47	41.5	1.10	1.9			
			50	3.632	64	61.1	1.18	2.0			
			75	3.620	97	96.3	1.22	2.1			
			100	3.618	112	120.6	1.32	2.3			
	110	0.3	25	3.618	20	27.2	0.46	0.8	856	-19.3	-2412
			50	3.614	38	56.2	0.50	0.9	945	-19.6	-2177
			75	3.611	51	75.6	0.50	0.9	1113	7.8	867
			100	3.609	58	86.2	0.50	0.9			
		0.7	25	3.617	20	25.6	1.00	1.8	817	-23.2	-1288
			50	3.615	35	50.3	1.12	2.0	876	-6.9	-345
			75	3.613	53	78.3	1.15	2.0	975	13.0	650
			100	3.610	67	100.0	1.16	2.0	1093	26.9	1345
		1.1	25	3.624	26	27.8	1.31	2.3	995	-10.9	-474
			50	3.618	39	48.1	1.50	2.6	1093	-2.0	-77
			75	3.616	61	80.2	1.60	2.8			
			100	3.614	91	126.9	1.70	3.0			
I-2	75	0.3	25	3.768	65	110.1	0.60	1.0			
			50	3.748	112	159.4	0.50	0.9			
			75	3.734	142	129.6	0.32	0.6			
			100	3.727	126	89.8	0.25	0.4			
		0.7	25	3.755	29	24.9	0.70	1.2			
			50	3.741	61	51.0	0.68	1.2			
			75	3.736	96	80.3	0.68	1.2			
			100	3.730	120	100.6	0.68	1.2			
		1.1	25	3.774	37	31.7	1.10	1.9			
			50	3.758	66	56.3	1.09	1.9			
			75	3.747	102	86.4	1.08	1.9			
			100	3.735	138	117.2	1.08	1.9			

(Continued)

* For material I at temperatures of 75 and 90°F the capacity of the wall friction equipment was exceeded.

** To compute shear values: $S_G = \frac{90p - 3.82F + N \cdot b}{28.26h} \frac{\theta_{max}}{\theta_o}$

† Values cannot be computed when wall friction is missing.

Table A5 (Concluded)

Test No.	Test Temp °F	Initial Angle deg θ_o	Vertical Press. psi P_v	Avg Height in.	Avg Roller Press. psi P_R	Uncorrected Shear, psi S_G	Max Gyratory Angle, deg θ_{max}	Measured Max Strain %	Wall Friction Force lb F	Shear Stress psi	Shear Modulus psi
I-2	90	0.3	25	3.729	41	38.7	0.33	0.6			
			50	3.723	67	63.3	0.33	0.6			
			75	3.718	85	80.5	0.33	0.6			
			100	3.716	94	89.2	0.33	0.6			
		0.7	25	3.730	32	30.7	0.78	1.4			
			50	3.722	42	42.4	0.81	1.4			
			75	3.720	70	78.5	0.90	1.6			
			100	3.719	77	94.7	0.99	1.7			
		1.1	25	3.734	39	36.8	1.20	2.1			
			50	3.725	53	51.8	1.23	2.1			
			75	3.723	72	78.2	1.36	2.4			
			100	3.720	95	105.7	1.40	2.4			
	110	0.3	25	3.720	24	30.3	0.44	0.8	748	-7.4	-925
			50	3.716	38	54.7	0.50	0.9	824	7.3	811
			75	3.713	48	70.7	0.51	0.9	938	15.2	1689
			100	3.710	47	77.7	0.57	1.0	1071	6.2	620
		0.7	25	3.721	19	21.8	0.92	1.6	900	-26.0	-1625
			50	3.718	36	48.5	1.08	1.9	938	-9.3	-489
			75	3.715	51	70.1	1.10	1.9	995	8.0	421
			100	3.712	67	92.3	1.10	1.9	1125	22.9	1205
		1.1	25	3.721	20	22.4	1.40	2.4	710	-4.9	-204
			50	3.718	41	51.8	1.58	2.8	786	16.3	582
			75	3.715	55	72.0	1.63	2.8	976	24.6	879
			100	3.716	75	102.3	1.70	3.0	1128	44.5	1483
I-3	110	0.3	25	3.723	20	23.6	0.41	0.7	900	-18.6	-2657
			50	3.718	32	44.3	0.48	0.8	995	-11.0	-1375
			75	3.712	44	61.0	0.48	0.8	1185	-6.4	-800
			100	3.709	58	80.5	0.48	0.8	1375	2.4	300
		0.7	25	3.718	23	25.7	0.90	1.6	862	-19.4	-1212
			50	3.714	45	56.5	1.01	1.8	976	0.9	50
			75	3.711	60	82.4	1.10	1.9	1166	10.4	547
			100	3.706	77	107.8	1.12	2.0	1280	27.5	1375
		1.1	25	3.722	33	32.1	1.23	2.2	1090	-7.3	-332
			50	3.716	48	54.3	1.42	2.5	1185	3.5	140
			75	3.713	66	79.2	1.50	2.6	1471	11.7	450
			100	3.710	98	125.2	1.60	2.8	1651	42.9	1532

Table A6

Gyratory Shear Data--II

Test No.	Initial Angle deg θ_o	Vertical Press. psi P_v	Avg Height in.	Avg Roller Press. psi P_R	Uncorrected Shear, psi S_G	Max Gyratory Angle, deg θ_{max}	Measured Max Strain %	Wall Friction Force lb F	Shear Stress psi	Shear Modulus psi
II-1	0.3	25	3.647	27	36.4	0.46	0.8	352	18.9	2,362
		50	3.638	44	58.2	0.45	0.8	481	33.5	4,188
		75	3.630	63	74.2	0.40	0.7	639	45.0	6,429
		100	3.621	82	96.9	0.40	0.7	817	58.7	8,386
	0.7	25	3.627	37	46.8	1.00	1.8	481	16.0	889
		50	3.615	65	80.2	0.97	1.7	678	40.3	2,371
		75	3.603	87	103.4	0.93	1.6	817	57.5	3,594
		100	3.589	119	140.4	0.92	1.6	975	86.9	5,431
	1.1	25	3.591	39	48.8	1.53	2.7	481	28.4	1,052
		50	3.576	72	89.3	1.51	2.6	609	63.4	2,438
		75	3.559	104	128.9	1.50	2.6	777	93.8	3,608
		100	3.540	134	164.8	1.48	2.6	925	122.1	4,696
II-2	0.3	25	3.690	27	37.5	0.48	0.8	322	20.7	2,588
		50	3.678	50	65.3	0.45	0.8	421	43.8	5,475
		75	3.667	64	84.0	0.45	0.8	520	57.3	7,162
		100	3.658	89	104.0	0.40	0.7	658	73.7	10,529
	0.7	25	3.658	34	43.6	1.02	1.8	342	19.9	1,106
		50	3.646	59	74.5	1.00	1.8	490	44.2	2,456
		75	3.633	83	103.1	0.98	1.7	668	63.1	3,712
		100	3.618	111	132.8	0.94	1.6	836	85.7	5,356
	1.1	25	3.624	40	49.2	1.52	2.7	431	31.8	1,178
		50	3.609	67	84.1	1.54	2.7	579	58.5	2,167
		75	3.591	95	118.4	1.52	2.7	708	87.2	3,230
		100	3.569	135	166.9	1.50	2.6	836	128.5	4,942
II-3 (Saturated)	0.3	25	3.583	29	43.2	0.50	0.9	303	45.2	5,044
		50	3.573	48	64.6	0.45	0.8	441	61.8	7,725
		75	3.565	73	91.8	0.42	0.7	520	86.5	12,357
		100	3.557	90	108.1	0.40	0.7	658	96.6	13,800
	0.7	25	3.568	35	48.7	1.08	1.9	382	40.8	2,147
		50	3.554	65	86.6	1.03	1.8	500	72.2	4,011
		75	3.542	91	118.2	1.00	1.8	609	99.0	5,500
		100	3.527	115	145.6	0.97	1.7	718	121.4	7,141
	1.1	25	3.540	38	50.4	1.60	2.8	342	50.9	1,818
		50	3.522	69	92.1	1.60	2.8	490	84.7	3,025
		75	3.504	98	131.7	1.60	2.8	639	115.3	4,118
		100	3.480	123	166.5	1.60	2.8	836	139.2	4,971
II-4 (Saturated)	0.3	25	3.588	26	38.7	0.50	0.9	303	41.0	4,556
		50	3.579	47	63.1	0.45	0.8	402	62.6	7,825
		75	3.570	65	81.7	0.42	0.7	481	77.6	11,086
		100	3.562	82	98.5	0.40	0.7	619	89.3	12,757
	0.7	25	3.572	26	36.2	1.08	1.9	322	31.4	1,653
		50	3.559	56	76.1	1.05	1.8	461	59.8	3,322
		75	3.549	74	98.2	1.02	1.8	540	82.6	4,589
		100	3.533	101	131.8	1.00	1.8	639	110.9	6,161
	1.1	25	3.544	34	45.7	1.62	2.8	362	45.7	1,632
		50	3.527	61	82.0	1.61	2.8	520	72.3	2,582
		75	3.509	83	112.4	1.61	2.8	698	92.5	3,304
		100	3.487	109	147.6	1.60	2.8	856	119.2	4,257

Table A7

Gyratory Shear Data--III

Test No.	Test Temp °F	Initial Angle deg θ_o	Vertical Press. psi P_v	Avg Height in.	Avg Roller Press. psi F_R	Uncorrected Shear, psi S_G	Max Gyratory Angle, deg θ_{max}	Measured Max Strain %	Wall Friction Force lb F	Shear Stress psi	Shear Modulus psi
III-1	75	0.3	25	3.732	19	23.4	0.43	0.8	164	16.6	2,075
			50	3.719	28	34.7	0.43	0.8	204	26.2	3,275
			75	3.706	38	47.4	0.43	0.8	243	37.2	4,650
			100	3.698	46	54.9	0.41	0.7	283	42.8	6,114
	75	0.7	25	3.703	26	27.5	0.85	1.5	85	19.0	1,267
			50	3.694	40	45.6	0.91	1.6	125	35.3	2,206
			75	3.688	54	61.1	0.90	1.6	144	49.9	3,119
			100	3.682	71	80.5	0.90	1.6	184	67.0	4,118
	75	1.1	25	3.696	24	23.1	1.20	2.1	85	24.9	1,186
			50	3.689	49	53.1	1.35	2.4	105	53.9	2,246
			75	3.682	64	72.3	1.40	2.4	125	72.0	3,000
			100	3.677	78	93.5	1.48	2.6	154	91.4	3,515
	90	0.3	25	3.676	12	14.7	0.42	0.7	95	11.9	1,700
			50	3.673	21	25.9	0.42	0.7	125	21.1	3,014
			75	3.670	27	33.4	0.42	0.7	144	27.6	3,943
			100	3.668	31	37.5	0.41	0.7	164	31.7	4,529
	90	0.7	25	3.676	23	26.6	0.92	1.6	85	17.7	1,106
			50	3.671	38	45.6	0.95	1.7	95	35.6	2,094
			75	3.668	50	63.3	1.00	1.8	115	52.1	2,894
			100	3.664	63	80.0	1.00	1.8	144	57.6	3,200
	90	1.1	25	3.677	23	26.0	1.40	2.4	95	26.4	1,100
			50	3.673	38	45.6	1.48	2.6	125	45.0	1,731
			75	3.668	48	58.8	1.50	2.6	144	57.2	2,200
			100	3.664	69	85.0	1.51	2.6	184	80.4	3,092
	110	0.3	25	3.663	11	16.2	0.50	0.9	85	12.6	1,400
			50	3.660	19	28.0	0.50	0.9	105	24.1	2,678
			75	3.658	27	36.6	0.46	0.8	125	31.2	3,900
			100	3.656	32	43.5	0.46	0.8	144	37.9	4,738
	110	0.7	25	3.664	20	26.7	1.06	1.9	85	16.8	884
			50	3.660	33	44.4	1.06	1.9	105	34.6	1,800
			75	3.656	48	62.3	1.02	1.8	135	49.8	2,767
			100	3.653	60	76.5	1.00	1.8	174	62.5	3,472
	110	1.1	25	3.667	24	30.0	1.55	2.7	95	30.7	1,137
			50	3.662	40	51.3	1.58	2.8	125	50.3	1,796
			75	3.657	55	70.9	1.58	2.8	164	67.4	2,407
			100	3.654	72	92.9	1.58	2.8	204	87.3	3,118
III-2	75	0.3	25	3.842	15	20.1	0.48	0.8	243	8.1	1,012
			50	3.829	26	35.0	0.48	0.8	283	20.7	2,588
			75	3.814	37	46.9	0.45	0.8	322	31.5	3,938
			100	3.801	45	57.3	0.45	0.8	382	38.5	4,812
	75	0.7	25	3.799	22	25.4	0.95	1.7	184	11.4	671
			50	3.789	38	41.9	0.90	1.6	204	27.3	1,706
			75	3.777	53	59.3	0.91	1.6	243	43.2	2,700
			100	3.764	66	74.2	0.91	1.6	283	56.5	3,531
	75	1.1	25	3.771	27	26.5	1.25	2.2	164	25.0	1,136
			50	3.764	45	47.9	1.35	2.4	204	44.5	1,712
			75	3.754	64	75.0	1.48	2.6	243	68.0	2,615
			100	3.744	82	97.7	1.50	2.6	293	88.9	3,419
	90	0.3	25	3.736	15	20.7	0.48	0.8	144	13.0	1,625
			50	3.732	23	29.8	0.45	0.8	184	22.4	2,800
			75	3.728	32	41.6	0.45	0.8	204	32.2	4,025
			100	3.725	36	42.8	0.41	0.7	243	32.8	4,686
	90	0.7	25	3.734	20	22.3	0.90	1.6	135	11.6	725
			50	3.728	34	39.8	0.95	1.7	154	28.3	1,665
			75	3.723	50	58.7	0.94	1.6	184	44.9	2,806
			100	3.718	67	76.2	0.91	1.6	224	61.2	3,825
	90	1.1	25	3.733	22	25.4	1.45	2.5	135	24.1	964
			50	3.725	43	51.4	1.50	2.6	184	47.1	1,812
			75	3.718	60	72.5	1.51	2.6	214	66.6	2,562
			100	3.711	75	91.0	1.51	2.6	263	83.4	3,208
	110	0.3	25	3.707	14	20.3	0.50	0.9	135	13.7	1,522
			50	3.702	23	30.1	0.45	0.8	174	23.1	2,888
			75	3.699	28	36.7	0.45	0.8	224	26.6	3,325
			100	3.696	33	40.5	0.42	0.7	253	30.1	4,300
	110	0.7	25	3.706	18	22.5	1.00	1.8	125	10.8	600
			50	3.701	31	37.0	0.95	1.7	164	23.7	1,394
			75	3.696	48	55.6	0.92	1.6	204	40.6	2,538
			100	3.692	61	69.2	0.90	1.6	224	53.9	3,369
	110	1.1	25	3.706	24	28.8	1.50	2.6	144	26.8	1,031
			50	3.700	41	49.4	1.50	2.6	184	45.5	1,750
			75	3.694	54	65.4	1.50	2.6	224	59.1	2,273
			100	3.688	71	85.1	1.48	2.6	273	77.2	2,969

(Continued)

Table A7 (Concluded)

Test No.	Test Temp °F	Initial Angle deg θ_o	Vertical Press. psi P_v	Avg Height in.	Avg Roller Press. psi P_R	Uncorrected Shear, psi S_G	Max Gyratory Angle, deg θ_{max}	Measured Max Strain %	Wall Friction Force lb F	Shear Stress psi	Shear Modulus psi
III-3 (Saturated)	75	0.3	25	3.829	23	25.7	0.40	0.7	243	36.4	5,200
			50	3.812	38	48.0	0.45	0.8	303	53.7	6,712
			75	3.797	50	63.3	0.45	0.8	352	66.1	8,262
			100	3.786	57	69.5	0.43	0.8	392	71.2	8,900
	75	0.7	25	3.797	27	26.9	0.82	1.4	164	34.0	2,429
			50	3.788	40	44.0	0.90	1.6	233	47.3	2,956
			75	3.779	54	60.4	0.91	1.6	273	62.0	3,875
			100	3.771	68	76.2	0.91	1.6	322	75.6	4,725
	75	1.1	25	3.788	30	23.4	1.00	1.8	174	37.4	2,078
			50	3.778	46	48.8	1.35	2.4	224	59.2	2,467
			75	3.772	64	68.1	1.35	2.4	263	76.8	3,200
			100	3.765	81	86.4	1.35	2.4	303	93.0	3,875
	90	0.3	25	3.761	20	27.3	0.48	0.8	174	39.6	4,950
			50	3.755	28	38.4	0.48	0.8	214	47.9	5,988
			75	3.752	39	50.2	0.45	0.8	243	58.9	7,362
			100	3.748	46	55.5	0.42	0.7	283	63.4	9,057
	90	0.7	25	3.763	23	27.7	0.98	1.8	135	35.3	1,961
			50	3.756	40	38.3	0.98	1.8	174	54.0	3,000
			75	3.751	55	63.9	0.94	1.6	224	67.1	4,194
			100	3.746	67	74.8	0.90	1.6	263	76.2	4,762
	90	1.1	25	3.764	25	28.5	1.45	2.5	154	41.5	1,660
			50	3.756	47	54.9	1.48	2.6	204	64.9	2,496
			75	3.750	62	72.8	1.48	2.6	253	80.8	3,108
			100	3.744	83	97.6	1.48	2.6	313	101.9	6,885
	110	0.3	25	3.739	21	30.0	0.50	0.9	164	41.5	4,611
			50	3.734	33	45.5	0.48	0.8	204	55.9	6,988
			75	3.731	44	56.9	0.45	0.8	224	67.0	8,375
			100	3.728	53	61.1	0.40	0.7	263	70.2	10,029
	110	0.7	25	3.744	22	28.5	1.05	1.8	164	33.2	1,844
			50	3.736	38	47.1	1.00	1.8	214	50.1	2,783
			75	3.732	50	59.2	0.95	1.7	273	59.3	3,488
			100	3.727	67	75.2	0.90	1.6	322	74.2	4,638
	110	1.1	25	3.745	26	30.8	1.50	2.6	164	43.0	1,654
			50	3.738	44	52.4	1.50	2.6	224	61.7	2,373
			75	3.731	63	74.3	1.48	2.6	283	80.0	3,077
			100	3.725	80	73.5	1.45	2.5	342	95.8	3,832
III-4 (Saturated)	75	0.3	25	3.913	23	26.4	0.42	0.7	411	27.9	3,986
			50	3.879	40	49.7	0.45	0.8	471	46.4	5,800
			75	3.849	51	63.9	0.45	0.8	540	56.8	7,100
			100	3.827	69	84.4	0.45	0.8	619	73.8	9,225
	75	0.7	25	3.834	26	25.0	0.80	1.4	293	27.2	1,943
			50	3.818	45	49.0	0.90	1.6	313	49.2	3,075
			75	3.802	61	70.6	0.95	1.7	382	66.5	3,912
			100	3.786	72	83.9	0.95	1.7	481	74.2	4,365
	75	1.1	25	3.800	29	27.0	1.20	2.1	253	36.8	1,753
			50	3.789	55	59.2	1.38	2.4	293	66.3	2,762
			75	3.776	74	85.9	1.48	2.6	352	88.5	3,404
			100	3.763	92	108.8	1.50	2.6	411	108.9	4,188
	90	0.3	25	3.755	24	30.7	0.45	0.8	243	39.0	4,875
			50	3.749	37	47.5	0.45	0.8	313	52.8	6,600
			75	3.744	48	57.7	0.42	0.7	362	60.8	8,686
			100	3.741	63	72.2	0.40	0.7	421	73.7	10,529
	90	0.7	25	3.755	22	25.7	0.95	1.7	224	28.8	1,694
			50	3.747	42	49.3	0.95	1.7	823	49.4	2,906
			75	3.740	53	60.5	0.92	1.6	322	58.8	3,675
			100	3.734	70	80.1	0.92	1.6	382	76.3	4,769
	90	1.1	25	3.753	29	32.2	1.41	2.5	243	40.5	1,620
			50	3.744	50	58.5	1.48	2.6	303	63.7	2,450
			75	3.733	71	83.4	1.48	2.6	372	85.5	3,288
			100	3.729	90	106.0	1.48	2.6	431	105.3	4,050
	110	0.3	25	3.722	23	31.7	0.48	0.8	204	41.2	5,150
			50	3.715	44	57.0	0.45	0.8	243	65.5	8,188
			75	3.713	52	67.5	0.45	0.8	273	93.0	11,625
			100	3.709	62	75.3	0.42	0.7	322	81.0	11,571
	110	0.7	25	3.724	25	32.5	1.05	1.8	214	34.9	1,939
			50	3.716	46	57.2	1.00	1.8	263	57.1	3,172
			75	3.710	65	77.0	0.95	1.7	313	75.5	4,441
			100	3.705	83	93.3	0.90	1.6	382	89.0	5,562
	110	1.1	25	3.724	28	33.3	1.50	2.6	224	41.7	1,604
			50	3.714	49	57.8	1.48	2.6	283	64.2	2,469
			75	3.708	70	82.9	1.48	2.6	342	86.0	3,308
			100	3.700	92	109.1	1.48	2.6	402	109.6	4,215

Table A8

Gyratory Shear Data--IV

Test No.	Initial Angle deg θ_o	Vertical Press. psi P_v	Avg Height in.	Avg Roller Press. psi P_R	Uncorrected Shear, psi S_G	Max Gyratory Angle, deg θ_{max}	Measured Max Strain %	Wall Friction Force lb F	Shear Stress psi	Shear Modulus psi
IV-1	0.3	25	3.698	24	27.7	0.40	0.7	214	19.4	2,771
		50	3.689	40	46.4	0.40	0.7	322	32.8	4,686
		75	3.680	57	63.0	0.38	0.7	451	43.7	6,243
		100	3.674	74	79.6	0.37	0.6	599	55.0	9,167
	0.7	25	3.685	29	32.6	0.90	1.6	224	17.6	1,100
		50	3.676	58	65.4	0.90	1.6	362	43.5	2,719
		75	3.666	81	90.2	0.89	1.6	481	63.3	3,956
		100	3.656	98	107.6	0.87	1.5	619	74.4	4,960
	1.1	25	3.690	33	34.8	1.32	2.3	253	29.2	1,270
		50	3.675	63	66.7	1.32	2.3	382	55.0	2,391
		75	3.665	93	98.7	1.32	2.3	520	80.5	3,500
		100	3.653	119	126.8	1.32	2.3	658	102.4	4,452
IV-2	0.3	25	3.453	24	34.9	0.47	0.8	184	25.7	3,212
		50	3.441	37	54.1	0.47	0.8	273	39.4	4,925
		75	3.432	52	68.1	0.42	0.7	342	51.3	7,328
		100	3.425	63	78.8	0.40	0.7	431	57.8	8,257
	0.7	25	3.444	30	36.9	0.92	1.6	224	20.9	1,306
		50	3.431	55	67.9	0.92	1.6	303	47.8	2,988
		75	3.421	73	88.6	0.90	1.6	402	63.7	3,981
		100	3.411	93	113.2	0.90	1.6	500	83.3	5,206
	1.1	25	3.445	34	42.1	1.45	2.5	224	35.7	1,428
		50	3.430	63	78.4	1.45	2.5	322	66.4	2,656
		75	3.418	87	108.8	1.45	2.5	421	92.4	3,696
		100	3.407	112	112.0	1.41	2.5	530	114.9	4,596
IV-3	0.3	25	3.623	23	29.2	0.43	0.8	194	21.3	2,662
		50	3.613	39	49.6	0.43	0.8	283	36.6	4,575
		75	3.605	54	64.1	0.40	0.7	392	47.0	6,714
		100	3.598	67	79.8	0.40	0.7	500	56.8	8,114
	0.7	25	3.623	31	35.8	0.91	1.6	184	22.5	1,406
		50	3.611	56	64.3	0.90	1.6	273	46.7	2,919
		75	3.599	79	91.1	0.90	1.6	362	69.2	4,325
		100	3.591	102	116.6	0.89	1.6	481	88.7	5,544
	1.1	25	3.624	40	45.3	1.40	2.4	253	38.2	1,592
		50	3.608	68	77.6	1.40	2.4	352	66.4	2,767
		75	3.599	94	107.7	1.40	2.4	461	91.2	3,800
		100	3.588	118	135.7	1.40	2.4	599	112.6	4,692
IV-4 (Saturated)	0.3	25	3.853	27	29.9	0.40	0.7	293	38.0	5,429
		50	3.839	41	45.7	0.40	0.7	392	49.5	7,071
		75	3.827	59	61.0	0.37	0.6	481	62.2	10,367
		100	3.819	73	71.6	0.35	0.6	589	69.4	11,567
	0.7	25	3.825	33	33.3	0.84	1.5	332	33.3	2,220
		50	3.812	58	57.5	0.82	1.4	461	52.7	3,764
		75	3.802	78	75.7	0.80	1.4	579	66.2	4,279
		100	3.791	97	94.5	0.80	1.4	708	80.0	5,714
	1.1	25	3.812	37	37.1	1.30	2.3	342	42.3	1,839
		50	3.797	67	67.5	1.30	2.3	441	68.6	2,983
		75	3.782	96	97.2	1.30	2.3	560	93.6	4,070
		100	3.767	118	120.2	1.30	2.3	698	110.5	4,804
IV-5	0.3	25	3.675	22	28.8	0.45	0.8	224	18.1	2,262
		50	3.669	38	48.8	0.44	0.8	313	33.5	4,188
		75	3.664	52	60.8	0.40	0.7	402	43.5	6,214
		100	3.658	64	69.4	0.37	0.6	520	47.4	7,900
	0.7	25	3.675	28	29.9	0.85	1.5	204	15.6	1,040
		50	3.667	49	50.6	0.82	1.4	293	33.1	2,364
		75	3.660	65	65.7	0.80	1.4	402	44.3	3,164
		100	3.653	85	86.1	0.80	1.4	500	60.1	4,293
	1.1	25	3.675	31	32.1	1.29	2.6	243	27.1	1,042
		50	3.664	57	58.8	1.28	2.2	352	48.9	2,223
		75	3.654	86	86.8	1.25	2.2	441	73.9	3,359
		100	3.644	105	106.5	1.25	2.2	540	89.2	4,055

Table A9

Gyratory Shear Data--V

Test No.	Initial Angle deg θ_o	Vertical Press. psi P_v	Avg Height in.	Avg Roller Press. psi P_R	Uncorrected Shear, psi S_G	Max Gyratory Angle, deg θ_{max}	Measured Max Strain %	Wall Friction Force lb F	Shear Stress psi	Shear Modulus psi
V-1	0.3	25	3.834	24	30.8	0.46	0.8	322	15.0	1,875
		50	3.825	39	50.2	0.46	0.8	441	28.0	3,500
		75	3.818	51	64.4	0.45	0.8	550	37.3	4,662
		100	3.808	66	78.0	0.42	0.7	658	47.0	6,714
	0.7	25	3.826	30	34.3	0.95	1.7	293	15.5	912
		50	3.815	54	60.0	0.92	1.6	372	37.9	2,379
		75	3.804	71	77.6	0.90	1.6	461	51.4	3,212
		100	3.790	92	101.0	0.90	1.6	570	70.4	4,400
	1.1	25	3.810	33	34.4	1.35	2.4	332	24.8	1,033
		50	3.796	58	65.4	1.45	2.5	431	50.8	2,032
		75	3.780	79	92.7	1.50	2.6	550	71.5	2,750
		100	3.758	102	120.5	1.50	2.6	698	91.8	3,531
V-2	0.3	25	3.616	18	27.2	0.51	0.9	233	14.0	1,556
		50	3.606	32	47.5	0.50	0.9	342	27.8	3,089
		75	3.598	46	65.8	0.48	0.8	431	41.5	5,188
		100	3.590	62	83.3	0.45	0.8	520	56.5	7,062
	0.7	25	3.602	26	34.6	1.04	1.8	253	15.2	844
		50	3.592	48	64.1	1.04	1.8	352	40.1	2,228
		75	3.581	71	95.2	1.04	1.8	471	64.4	3,578
		100	3.568	90	116.5	1.00	1.8	589	79.8	4,433
	1.1	25	3.586	34	42.4	1.52	2.7	303	31.8	1,178
		50	3.571	64	80.2	1.52	2.7	402	64.0	2,370
		75	3.556	83	108.2	1.57	2.7	481	86.8	3,215
		100	3.539	99	131.6	1.59	2.8	570	104.8	3,743
V-3	0.3	25	3.566	22	34.3	0.52	0.9	204	22.5	2,500
		50	3.557	36	53.1	0.49	0.9	313	36.2	4,022
		75	3.549	54	68.4	0.42	0.7	392	49.2	7,029
		100	3.542	66	83.8	0.42	0.7	500	59.2	8,457
	0.7	25	3.551	25	33.1	1.02	1.8	283	13.1	728
		50	3.543	47	61.2	1.00	1.8	402	34.9	1,939
		75	3.532	64	83.7	1.00	1.8	560	47.9	2,661
		100	3.522	79	100.7	0.97	1.7	639	61.5	3,618
	1.1	25	3.536	32	40.0	1.50	2.6	382	25.2	969
		50	3.526	58	73.2	1.51	2.6	530	50.5	1,942
		75	3.512	83	105.3	1.51	2.6	649	76.2	2,931
		100	3.449	106	135.1	1.51	2.6	817	96.5	3,712
V-4 (Saturated)	0.3	25	3.602	19	30.5	0.54	0.9	224	37.1	4,122
		50	3.592	34	50.7	0.50	0.9	332	52.1	5,789
		75	3.584	43	60.5	0.47	0.8	411	58.5	7,312
		100	3.576	55	74.3	0.45	0.8	500	67.7	8,462
	0.7	25	3.586	21	29.0	1.07	1.9	283	26.7	1,405
		50	3.574	40	54.9	1.06	1.8	411	44.9	2,494
		75	3.599	58	80.0	1.06	1.8	540	62.9	3,494
		100	3.550	85	110.7	1.06	1.8	629	90.5	5,028
	1.1	25	3.565	24	31.9	1.60	2.8	332	33.9	1,256
		50	3.551	52	71.4	1.65	2.9	411	67.4	2,324
		75	3.536	76	101.7	1.60	2.8	500	93.8	3,350
		100	3.518	98	127.7	1.55	2.7	609	114.2	4,230
V-5 (Saturated)	0.3	25	3.618	15	24.4	0.55	1.0	283	26.9	2,690
		50	3.610	34	50.4	0.50	0.9	451	44.1	4,900
		75	3.601	47	62.9	0.45	0.8	550	53.8	6,725
		100	3.593	64	85.9	0.45	0.8	649	70.6	8,825
	0.7	25	3.604	23	29.5	1.00	1.8	293	27.8	1,544
		50	3.592	42	56.8	1.05	1.8	382	49.7	2,761
		75	3.581	57	77.4	1.05	1.8	481	63.7	3,539
		100	3.568	73	99.6	1.05	1.8	609	78.5	4,361
	1.1	25	3.586	28	34.6	1.50	2.6	342	36.5	1,404
		50	3.570	49	65.0	1.60	2.8	481	58.0	2,071
		75	3.544	68	90.7	1.60	2.8	599	77.0	2,750
		100	3.536	81	115.7	1.70	3.0	708	93.8	3,127

APPENDIX B: TRIAXIAL TESTS

General

1. The triaxial tests were conducted as supplementary testing to the gyratory testing and as such the extent of testing was very limited. Monies were available for the preparation and testing of only a single test specimen for each material, and thus test procedures were used to obtain maximum information from the single specimen. The testing was conducted by personnel of the Soils Research Facility.

Materials

2. Of the five materials tested in the GTM, only four were selected for triaxial testing: a dense bituminous-stabilized base course; a crushed aggregate identified as NJDOT base 5A; a bituminous-stabilized open-graded base material; and a nonstabilized open-graded base material. In the gyratory testing these materials had been designated as materials I, V, III, and II, respectively.

3. Test specimens of the nonstabilized materials, i.e., materials V and II, were compacted by impact compaction in seven 2-in. layers. The compaction effort used was the effort necessary to obtain densities approximating the densities obtained in the gyratory testing. The physical data for the specimens are given in Table B1.

4. The specimens of the bituminous-stabilized bases were prepared utilizing static compaction. As with the nonstabilized specimens, the target densities were the same as those obtained in the gyratory testing machine. The compaction force used was necessary to obtain the target density. The specimen data are contained in Table B1.

Test Equipment and Procedure

Testing equipment

5. The test equipment used in this testing program was essentially

the same as the equipment described in Reference 20. Basically, the equipment consisted of a conventional triaxial cell, a closed-loop electrohydraulic loading system, a miniature electronic load cell, and an arrangement of linear variable differential transformers (LVDT) for measuring specimen deformation. The loading system contained a function generator such that the load could be programmed for application in standard functions. The miniature load cell was inside the triaxial cell to insure that accurate measurements of applied load were obtained. The LVDT's for measuring deformations were attached to the specimens by two circular clamps. The clamps were placed at approximate thirds of the specimen, thus the axial deformation was over the center third of the specimen. The axial deformation was the average of the deformations occurring at each point. A schematic of the equipment for the repeated load triaxial testing is given in Figure B1.

Testing procedure

6. The testing procedure used was aimed first at obtaining information on the relative rutting potential of the different materials and second at the resilient deformation characteristics of each material. To accomplish these aims, each specimen was first subjected to 10,000 load repetitions of a fixed loading that would cause measurable permanent strain. The axial load applied was programmed to be a haversine stress-time wave form for a 0.2-sec load duration at 2-sec intervals. A second test of 10,000 load repetitions of a different loading was also conducted. During the application of the repetitive load, both the resilient and permanent deformations were monitored. Additional tests were conducted for each material to better define the relationships of state of stress on the resilient properties of each material. For these additional tests, various combinations of axial and confining stress were applied until it was felt the resilient properties had been defined for that state of stress. The sequence of the loading is given in Tables B2-B5. It was noted that for the bituminous samples the testing with the long-term cycling was not the first test performed. This resulted from the fact that several loadings had to be applied before a loading was found that caused measurable permanent deformation.

7. After the repeated loading was conducted, the unstabilized specimens were loaded to failure in the manner of a conventional triaxial test. The bituminous-stabilized specimens were not tested to failure because the failure loads exceeded the load capacity of the test equipment.

Results

8. The condensed results of the repetitive tests are given in Tables B2-B5. Figure B2 presents the results for the nonstabilized materials of the conventional triaxial test. Additional information for the tests involving the permanent deformation is provided by Figures B3-B6.

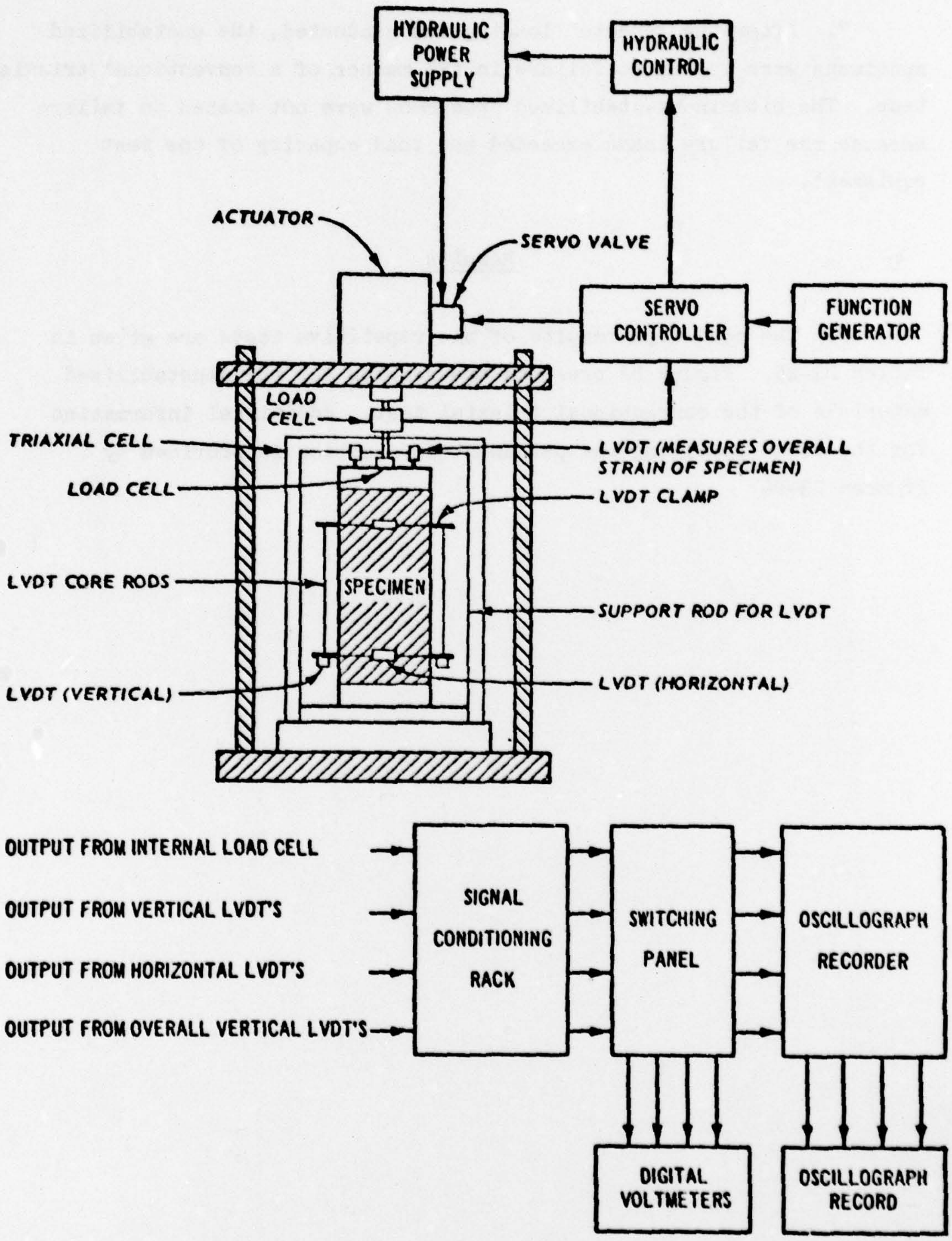

Figure B1. Schematic of the electronic control of loading pistons and the electronic instrumentation of the specimen (after Reference 20)

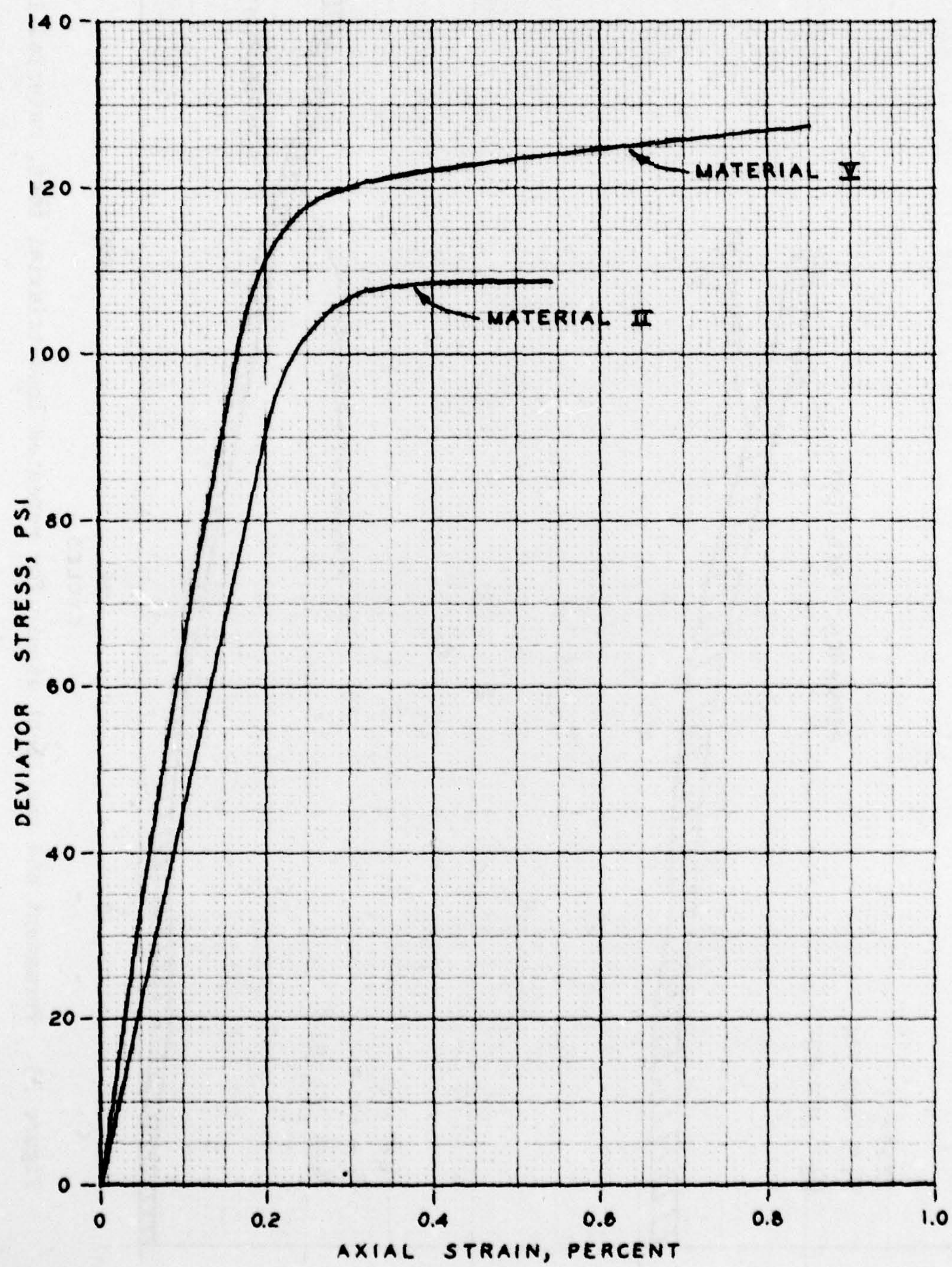

Figure B2. Stress-strain curves for conventional triaxial shear test with confining stress of 14.5 psi

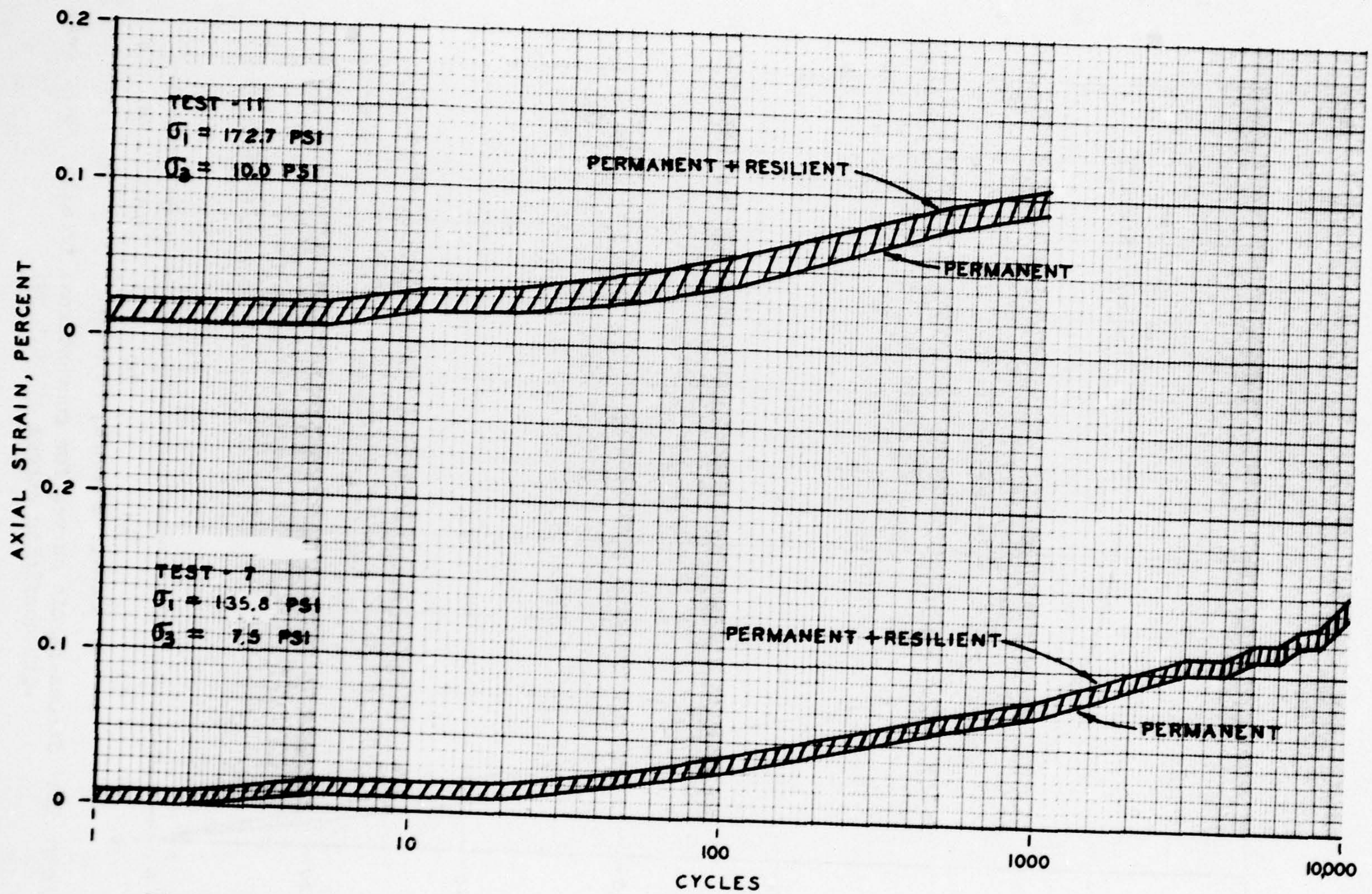

Figure B3. Permanent and resilient strain for repeated load triaxial test, material I

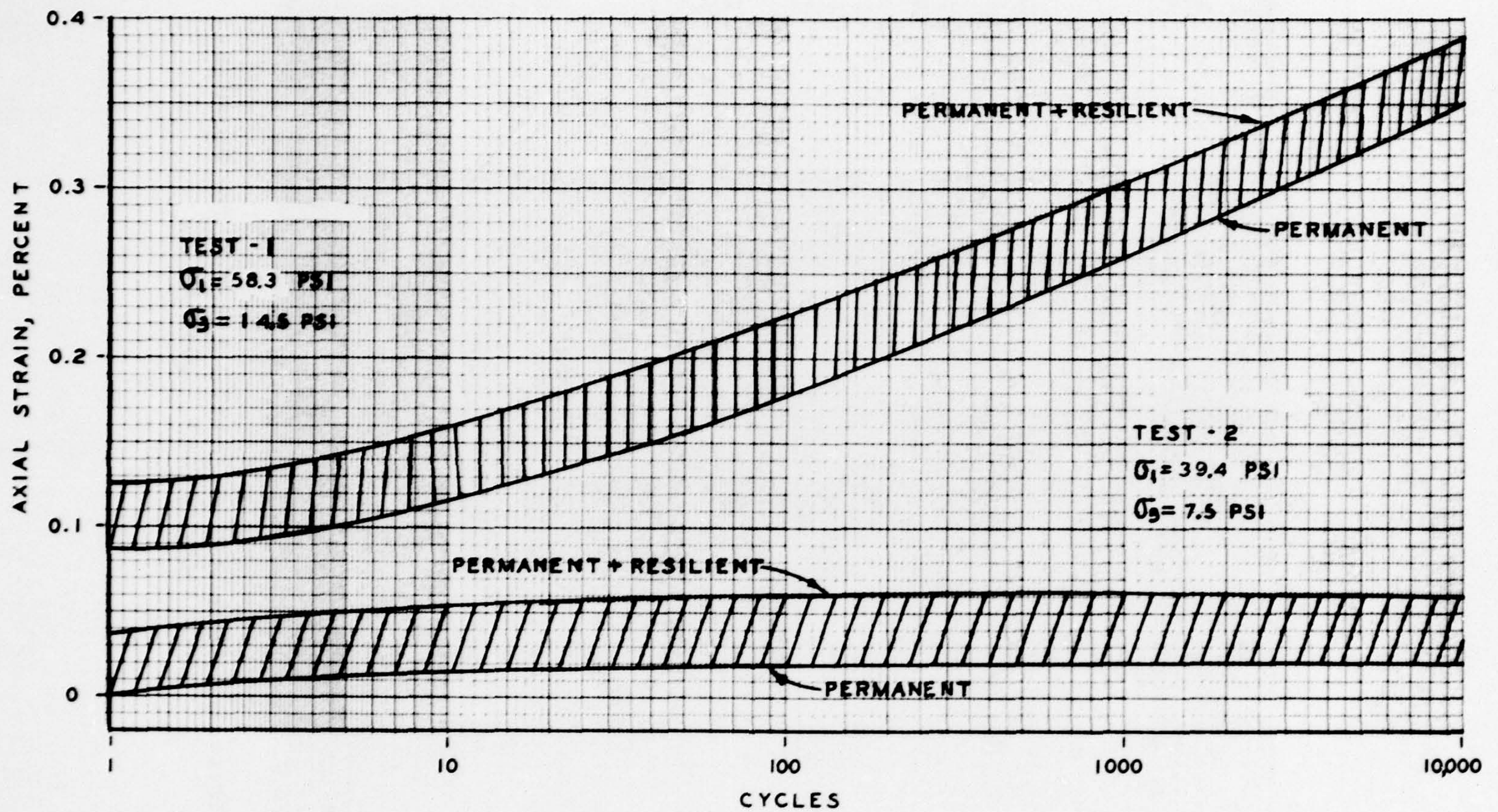

Figure B4. Permanent and resilient strain for repeated load triaxial test, material II

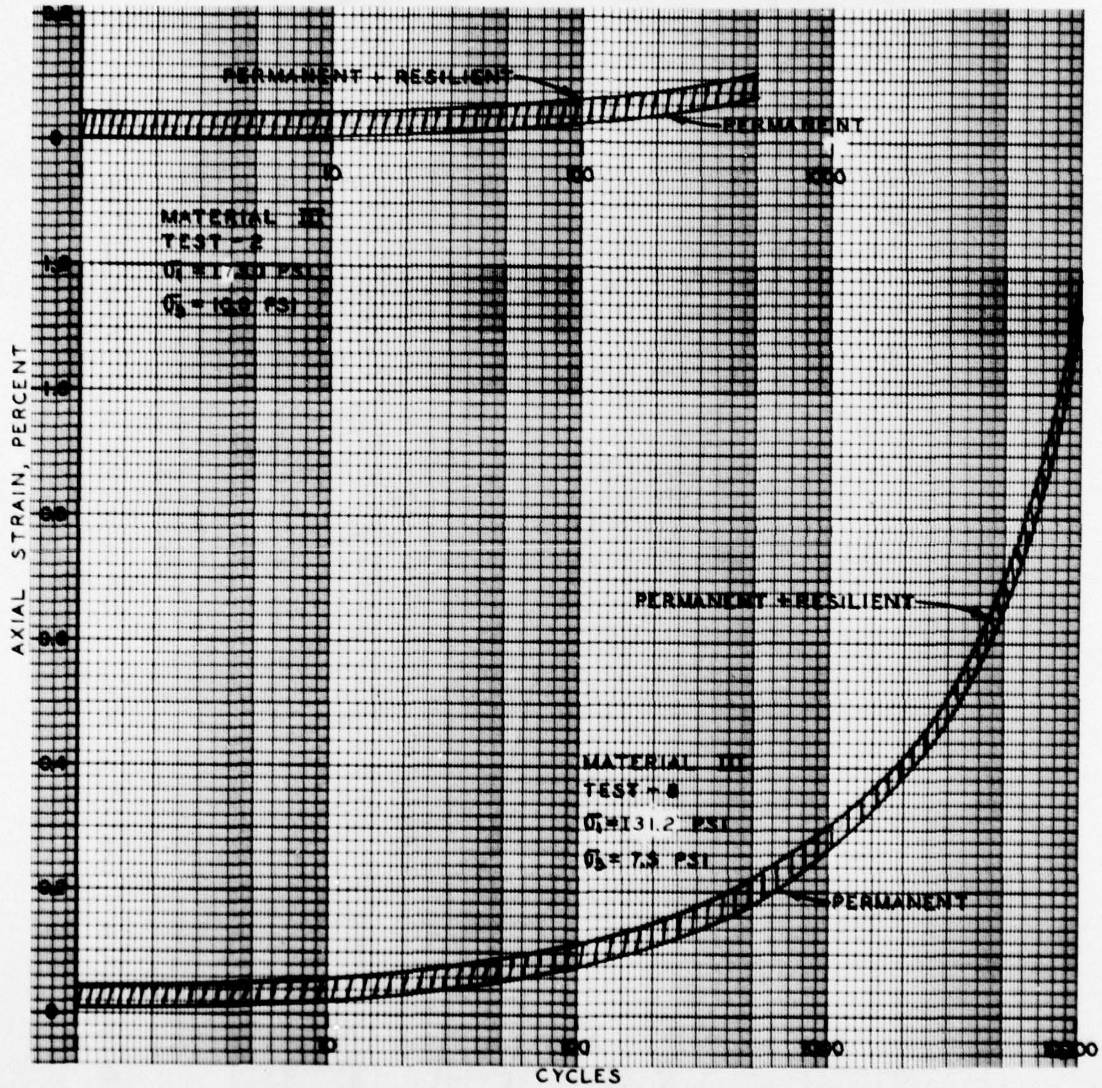

Figure B5. Permanent and resilient strain for repeated load triaxial test, material III

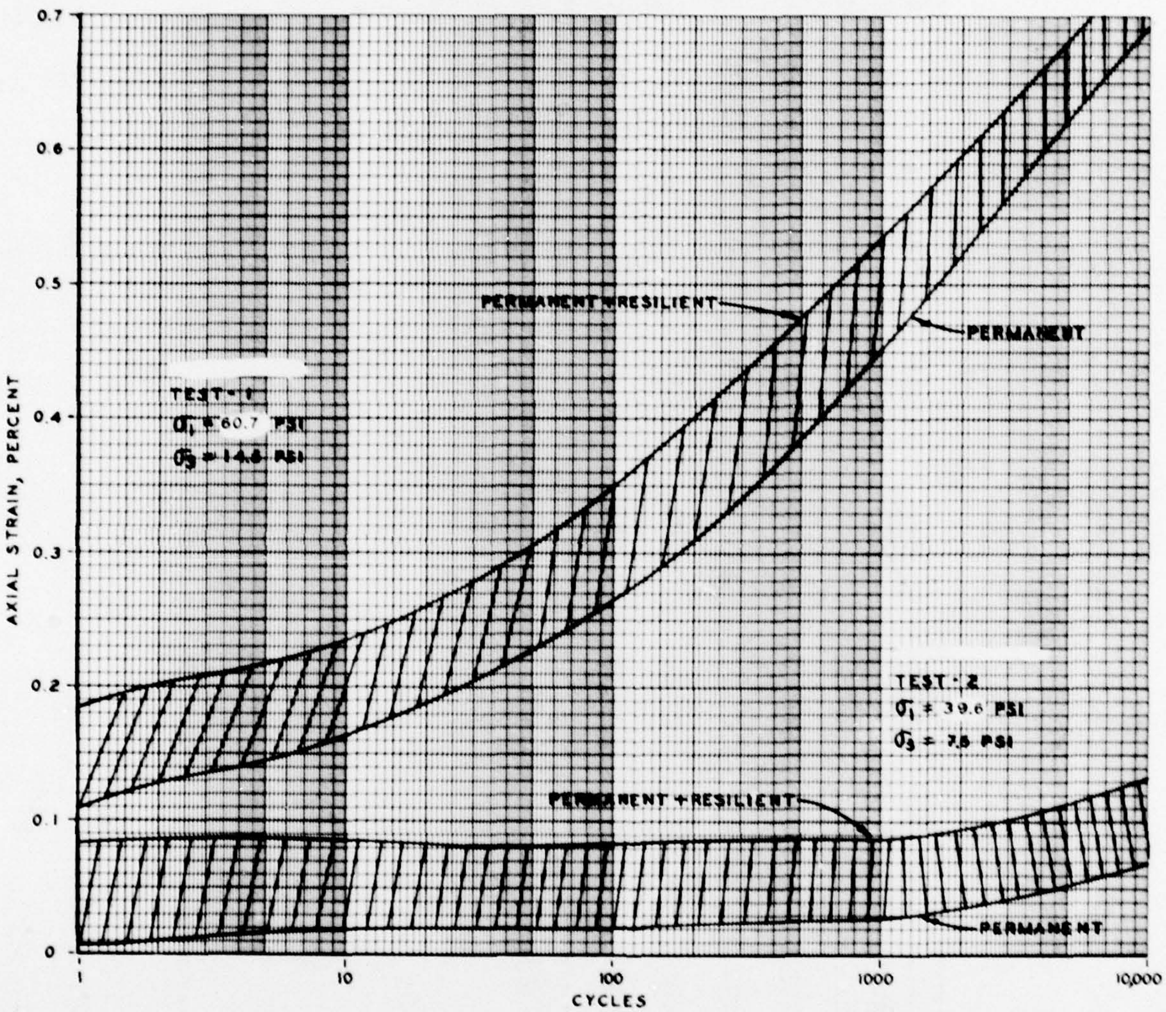

Figure B6. Permanent and resilient strain for repeated load triaxial test, material V

Table B1

Sample Data

Material	Sample Height in.	Diameter in.	Area $in.^2$	Volume $in.^3$	Weight Solids lb	Water Content %	Density pcf	Asphalt Content %
V	12.03	5.871	27.08	325.71	26.81	6.0	142.26	--
II	11.91	5.892	27.27	324.64	20.48	1.0	109.01	--
I	13.08	6.077	29.00	379.42	32.98	--	150.20	4.8
III	13.05	6.104	29.26	381.88	26.95	--	121.95	3.0

Table B2

Summary of Triaxial Compression Test Data for Material V (Nonstabilized) Samples

Material	Test No.	Chamber Pressure psi	Axial Stress psi	Cycle	Resilient Modulus psi	Poisson's Ratio	Permanent Strain %
V	1	14.5	46.2	1	61,479	0.23	0.11
				100	57,544	0.23	0.27
				1,000	65,640	0.27	0.44
				10,000	76,344	0.25	0.69
V	2	7.5	32.1	1	42,507	0.45	0.01
				100	53,122	0.28	0.02
				1,000	53,113	0.28	0.03
				10,000	53,082	0.28	0.07
V	3	10.0	41.5	1	64,014	0.30	0.03
				10	61,016	0.29	0.03
				50	61,014	0.29	0.03
V	4	20.0	83.0	1	84,479	0.29	0.02
				10	73,185	0.27	0.06
				50	68,504	0.29	0.18
V	5	20.0	91.8	1	67,379	0.28	0.01
				10	69,929	0.31	0.05
				50	64,789	0.30	0.22
V	6	20.0	101.3	1	66,778	0.31	0.01
				10	62,546	0.32	0.07
				50	64,344	0.38	0.35

Table B3

Summary of Triaxial Compression Test Data for Material II (Nonstabilized) Samples

Material	Test No.	Chamber Pressure psi	Axial Stress psi	Cycle	Resilient Modulus psi	Poisson's Ratio	Permanent Strain %
II	1	14.5	43.8	1	97,167	0.36	0.09
				100	91,961	0.32	0.18
				1,000	109,112	0.32	0.27
				10,000	109,041	0.27	0.35
II	2	7.5	31.9	1	74,714	0.32	0.00
				100	79,374	0.34	0.01
				1,000	84,656	0.32	0.01
				10,000	90,688	0.29	0.03
II	3	10.0	39.6	1	105,071	0.27	0.00
				10	105,071	0.27	0.00
				50	105,071	0.27	0.00
II	4	20.0	77.2	1	102,365	0.22	0.04
				10	95,952	0.21	0.06
				50	85,266	0.21	0.08
II	5	14.5	77.6	1	73,097	0.24	0.00
				100	69,455	0.28	0.07
				1,000	70,825	0.30	0.44
II	6	7.5	37.7	1	62,189	0.34	0.00
				100	82,903	0.39	0.01
				1,000	87,767	0.36	0.02

Table B4

Summary of Triaxial Compression Test Data for Material I (Bituminous-Stabilized) Samples

Material	Test No.	Chamber Pressure psi	Axial Stress psi	Cycle	Resilient Modulus psi	Poisson's Ratio	Permanent Strain %
I	1	14.5	41.3	1	1,651,552	--	0.00
				100	1,651,565	--	0.01
				1,000	1,651,428	--	0.01
				6,000	1,651,333	--	0.01
I	2	7.5	38.7	1	1,548,008	0.66	0.02
				50	1,547,958	0.66	0.02
				200	1,547,958	0.66	0.02
I	3	7.5	54.2	1	2,167,768	0.66	0.00
				50	2,167,642	0.66	0.00
				200	2,167,517	0.66	0.01
I	4	7.5	70.6	1	1,410,885	0.33	0.00
				50	1,410,651	0.33	0.01
				200	1,410,523	0.33	0.01
I	5	7.5	84.3	1	1,686,079	0.33	0.00
				50	1,685,800	0.33	0.01
				200	1,685,562	0.33	0.02
I	6	7.5	98.6	1	1,314,483	0.44	0.00
				50	1,314,222	0.44	0.01
				200	1,313,993	0.44	0.02
I	7	7.5	128.3	1	1,281,789	0.49	0.00
				100	1,281,057	0.49	0.03
				1,000	1,279,997	0.49	0.07
				10,000	1,022,725	0.39	0.14
I	8	7.5	144.8	1	1,445,225	0.33	0.00
				10	1,445,142	0.33	0.01
				100	1,445,105	0.33	0.01
I	9	7.5	153.4	1	1,531,224	0.33	0.00
				10	1,531,186	0.33	0.01
				100	1,531,047	0.33	0.01
I	10	3.3	58.6	1	1,170,098	--	0.00
			55.2	100	2,202,410	--	0.00
			55.2	1,000	2,202,265	--	0.00
			55.2	10,000	1,100,933	--	0.01
I	11	10.0	153.4	1	1,020,672	0.44	0.01
			153.4	100	874,332	0.38	0.04
			162.7	1,000	926,581	0.38	0.09

Table B5

Summary of Triaxial Compression Test Data for Material III (Bituminous-Stabilized) Samples

Material	Test No.	Chamber Pressure psi	Axial Stress psi	Cycle	Resilient Modulus psi	Poisson's Ratio	Permanent Strain %
III	1	14.5	41.4	2	551,785	--	0.01
				50	551,716	--	0.02
				200	827,553	--	0.03
III	2	10.0	163.0	1	402,029	0.24	0.00
				50	379,144	0.23	0.02
				500	379,063	0.23	0.07
III	3	7.5	37.6	1	752,119	--	0.00
				50	752,051	--	0.00
				200	752,051	--	0.00
III	4	7.5	51.8	1	690,480	--	0.00
				50	517,800	--	0.01
				200	690,360	--	0.01
III	5	7.5	65.0	1	649,333	--	0.00
				50	519,389	--	0.01
				200	519,333	--	0.02
III	6	7.5	79.3	1	528,457	--	0.00
				50	452,847	--	0.01
				200	452,784	--	0.02
III	7	7.5	94.3	1	471,324	0.08	0.00
				50	418,837	0.07	0.02
				200	417,189	0.07	0.03
III	8	7.5	123.7	1	352,981	0.09	0.01
				100	328,059	0.13	0.08
				1,000	305,408	0.12	0.28
				10,000	228,873	0.18	1.13
III	9	3.3	52.6	1	296,958	--	0.01
			53.9	100	355,256	--	0.01
			55.6	1,000	366,346	--	0.01
			55.1	10,000	435,619	--	0.01

(Continued)

Table B5 (Concluded)

Material	Test No.	Chamber Pressure psi	Axial Stress psi	Cycle	Resilient Modulus psi	Poisson's Ratio	Permanent Strain %
III	10	3.3	60.9	1	801,622	--	0.00
				10	601,197	--	0.00
				100	604,528	--	0.00
III	11	3.3	74.5	1	588,907	--	0.00
				10	588,888	--	0.00
				100	490,740	--	0.00
III	12	3.3	81.6	1	537,372	--	0.00
				10	460,604	--	0.00
				100	460,536	--	0.01
III	13	3.3	86.8	1	571,738	--	0.00
				10	490,033	--	0.00
				100	490,017	--	0.00
III	14	3.3	93.7	1	462,900	--	0.00
				10	462,885	--	0.00
				100	463,679	--	0.01
III	15	3.3	106.8	1	469,093	0.07	0.00
				10	469,066	0.07	0.00
				100	469,009	0.07	0.01
III	16	3.3	112.6	1	494,177	0.07	0.00
				10	444,719	0.06	0.00
				100	444,653	0.06	0.01
III	17	3.3	118.8	1	469,297	0.13	0.00
				10	469,270	0.13	0.00
				50	426,546	0.12	0.01
III	18	3.3	125.5	1	450,706	0.18	0.00
				10	450,665	0.18	0.01
				100	450,517	0.18	0.02
III	19	3.3	132.0	1	432,266	0.16	0.00
				10	432,272	0.16	0.01
				100	399,896	0.20	0.02
III	20	3.3	139.2	1	392,652	0.23	0.00
				10	392,604	0.23	0.01
				200	365,996	0.22	0.05

APPENDIX C: NOTATION

b	arm of vertical force couple = $h \cdot \tan \theta_o$
c	cohesion, lb
D_f	depth of footing, ft
E_G	gyratory modulus of elasticity
F	force caused by wall friction, lb
G_G	gyratory shear modulus, psi
h	height of sample, ft
K_p	coefficient of passive earth pressures
M_R	resilient modulus
N	normal vertical load on specimen, psi
N_c, N_q, N_γ	bearing capacity factors
p	gage pressure for upper roller, psi
P_R	average roller pressure, psi
P_v	applied vertical pressure, psi
q	bearing capacity per unit of area
r	radius of the footing, ft
S_G	gyratory shear strength, psi
ϕ	angle of internal friction, deg
ϕ_G	angle of internal friction determined by GTM, deg
γ	unit weight of soil, lb
ν	Poisson's ratio
θ_{max}	maximum gyratory angle, deg
θ_o	initial gyratory angle, deg
σ	computed stress, psi
σ_1	major principal stress, psi
σ_3	minor principal stress, psi

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Barker, Walter R

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